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Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

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Journal of the
SANITARY ENGINEERING DIVISION
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RUSSIAN WATER SUPPLY AND TREATMENT PRACTICES[a]

By V. J. Calise[1] and W. A. Homer[2]

SYNOPSIS

Russia has many lakes, rivers, and streams, and generally has an abundant water supply. This paper reports on a survey of facilities in Russia and Eastern Europe with emphasis on municipal treatment practices and supplies. The technical literature and reports from visitors to Russia provided the main sources of information.

INTRODUCTION

Purpose of Paper and Sources of Information.—This paper is the result of an attempt to survey water-treatment practices in Russia and Eastern Europe with emphasis on municipal practices and water supplies. In preparing this paper, the writers had available three sources of information:

1. The technical literature from Russian periodicals, almost all of which required original translation and review for preparation of this paper. More than 150 literature references were reviewed, of which approximately 30 technical articles were translated into English.
2. Information from American and other visitors to Russia.
3. Information directly from Russian experts and authorities in this field. Direct communication was established with a number of these specialists.

Note.—Discussion open until August 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SA 2, March, 1960.

[a] Presented at the May 1959 Convention in Cleveland, Ohio.

[1] General Sales Manager Graver Water Conditioning Co. New York, New York.
[2] Senior Process Engineer Graver Water Conditioning Co. New York, New York.

The technical literature and reports from visitors to Russia provided the main sources of information.

Because of the relatively limited amount of up-to-date literature available and the failure of direct communication and a personal visit to Russia, (which had been scheduled for the summer of 1958 and had to be cancelled because of business commitments), the information contained herein is not complete nor does it necessarily represent the most up-to-date thinking and practices in all areas covered in this report. Nevertheless, the picture drawn from the available data of the state of the art in this field in Russia is, in the main, an accurate indication of what is to be found there.

Literature and data available from East European countries was much more limited and a study of this literature showed less progress than in Russia.

Various Aspects Investigated.—It was the purpose of the writers to obtain data concerning:

a. The nature of various water supplies in Russia and Eastern Europe.

b. The general prevalence and use of, as well as the relative emphasis on, various water treatment processes, equipment designs and chemical results for application in treatment of municipal (potable) supplies.

c. The engineering and economic evaluation of the various processes cited above, with respect to -

(1) Chemical costs and availability of various chemicals;

(2) Material costs and availability of construction materials; and

(3) The effect of centralized standards (if any) on equipment designs, and treatment results for various water uses (potable, boiler feed, etc.) and the prevalence and use of centralized public health standards for total dissolved solids, sterilization, total hardness, alkalinity, et al.

NATURE OF WATER SUPPLIES IN RUSSIA

In general, Russia is a land of many lakes, rivers and streams and has an abundant water supply in virtually all areas. Larger cities are located close to fairly abundant surface water supplies. Industry, including central station utilities, purchases water from municipal water treatment plants which are generally of large size and, in the main, are conventional sedimentation-sand filtration plants. Water treatment facilities such as power and steam utilities are planned for centralized design and control. The use of underground aquifers is not as prevalent in the U.S.S.R. as in Europe and in the United States. Table 1 shows five typical water supplies available in the Soviet Union. According to data published by the Hydrometerological Service, "low mineral" content waters of the type in Col. 2, 3 and 4 in Table 1, in which bicarbonates predominate over sulfates and chlorides, constitute about 80% of all rivers and lakes in the U.S.S.R. The high solids waters shown in Col. 5 were sulfates and chlorides predominate over bicarbonates are concentrated in the Donbas, Azov Sea, and Kazakhstan regions.

The frequent use of the work "demineralization" in Soviet literature as applied to partial solids reduction by lime treatment and split stream processes, as well as to full "demineralization," emphasizes the fact that the general run of high-hardness, high-bicarbonate waters in the U.S.S.R. lend themselves well to substantial solids reduction (usually 60-70%) by simple lime treatment or split-stream treatment. As with most surface supplies, seasonal variations introduce problems of dosage control and, in the case of split stream treatment, may reduce plant design capacities.

The practice of locating power plants and industries near the source of coal or other raw materials often "fixes" the water supply and requires the use of rather complex treatment plants to render the raw water suitable for use as makeup to high pressure boilers. The nature of the water supplies has also been a factor in the design of high pressure boilers (1,600 to 2,700 psig) resulting in virtual standardization in government prescribed multi-stage, multi-drum boilers with two and three stages of evaporation; and also resulting in requirements for steam washing with very high relative amounts of feedwater or condensate.

GENERAL INDUSTRIAL WATER TREATMENT PRACTICES

The heavy industries in the U.S.S.R. are located, of necessity, in the same general regions as the electric-generating facilities. Many steel and metallurgical plants were moved during World War II to the Ural Mountains, where

TABLE 1.—TYPICAL NATURAL WATERS OF THE USSR

Constituents (1)	Low (2)	Medium (3)	High (4)	Medium-High (5)	Very High (6)
Total Hardness (ppm as CaCO)	125	268	445	715	1070
M Alkalinity[a] (ppm as CaCO)	108	232	268	268	320
Ratio Ca to Mg	2.47	2.01	4.00	1.65	2.34
Sulfate SO_4 as $CaCO_3$	17.5	35.5	176	450	750
Chloride Cl_4 as $CaCO_3$	17.5	26.5	90	357	535
Silica as SiO_2	6	10	16-18	20-25	25-30

[a] M Alkalinity in all cases is bicarbonate alkalinity.

a heavy concentration of industry still remains. Much of the oil and chemical industry is concentrated in the Donbas-Black Sea area. The nature of the water supplies is similar to those shown in Table 1.

These surface supplies are treated by the various methods described subsequently according to the quality requirements of the particular industry. Most of the industrial plants built since 1950 have centralized facilities for water treatment. A typical metallurgical plant would treat a surface supply by coagulation and filtration to provide the bulk of its requirements for cooling water and other general uses. The remainder would be softened, usually by ion exchange, and deaerated for use in steam boilers, evaporative cooling of furnace walls, or for other high temperature cooling purposes.

More recently (as of 1960), the practice of centralized deaeration of all softened water, with regenerative cooling of the softened water going to distant locations, has become popular, particularly in steel plants.

The writers were unable to find extensive information concerning the treatment of process water for chemical plants or cooling water.

Many chemical and paper plants in the U.S.S.R. use large quantities of demineralized water in their process as well as for boiler feedwater, makeup

treatment to high pressure boilers up to 1,650 psig and 60% makeup. One source indicated that demineralized water was used by the Voronezh Radio Component Works in the production of various electronic components. The writers suspect that the relatively high cost of anion resins and the lack of experience with demineralization has restricted its use for the present to critical applications such as makeup to very high pressure central station boilers and electronic or transistor manufacture.

MUNICIPAL WATER TREATMENT

Coagulation and sand filtration of the relatively abundant surface waters appears to supply the bulk of potable water needs for the large cities. The equipment used for this purpose, is, for the most part, conventional coagulation basins with about 4-hr detention, followed by sand and gravel gravity filters. The usual coagulants are used (alum or iron salts) with lime for pH correction. Chlorination is not as widely used as in the United States, but recent trends and the development of gaseous chlorine-feeding equipment indicate a future increase in the use of chlorine.

Chlorine usage is apparently limited because of high gaseous chlorine and sodium hypochlorite cost as well as lack of standards, such as A.P.H.A. or U.S.P.H.S., relative to sterilization by use of chlorine, ozone, or other materials and methods. Organic and B.O.D. reduction by simple clarification and filtration is generally employed.

Improvements in Coagulation Techniques.—The "divided coagulation treatment" of potable water supplies is not unknown in the United States and in Europe, but Russian water-treatment specialists have investigated this method on a large scale and are recommending its use, where applicable.

The "divided coagulation treatment" (sometimes known as concentrated dosage method) consists of dividing the raw water into approximately equal parts and feeding the total required coagulant dosage into one stream. This is comparable to the Hoover split-line treatment so successfully employed over the years in many municipal plants in the United States (1).[3]

After the normal flash mixing and coagulation process, the two streams (one stream untreated and the other with double the normal dosage) are recombined in the sedimentation basins where the normal sedimentation process takes place.

Several tests at different locations (2) (3) have apparently indicated that at low temperatures (1°C to 12°C) the divided method of treatment has some definite advantages with respect to shorter settling time and better sedimentation coefficient.

At higher temperatures (12°C to 20°C) the beneficial effect of divided treatment is not marked, although slight advantages still appear as noted in Fig. 1.

Improvements in Equipment Designs.—Using the conventional basins (4-hr detention) considerable savings in area and concrete were claimed by designing two-floor settling basins. The first such design at the Gorkii Water Supply was a series design, in which the water flowed into the lower section, along the length of the section and reserved direction flowing back along the upper floor to the clear water outlet.

A later design at the Moscow Water Works was a parallel-flow type in which sedimentation took place simultaneously in both upper and lower sections, the flow being in one direction only.

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[3] "The investigation of Two-Floor Water Conducting Settlers," by Yu. B., Bagotski, E. V. Sizova, Vodosnakzhemie i Sanit. Techn., No. 4, Pages 6-9, 1956.

Viewing and sampling tubes were provided through the floor of the top basin for checking performance of the bottom basin as shown in Fig. 2.

Extensive tests were made on the installation at the Moscow Water Works, which has a capacity of 6,900 cu m per hr. The tests, including a chloride-

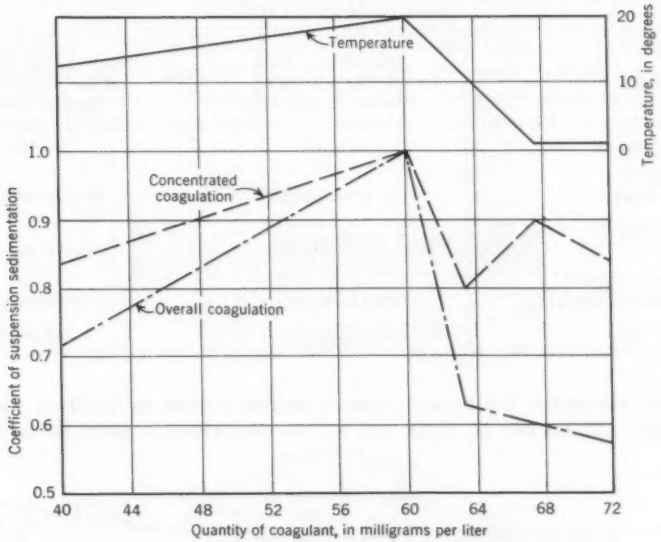
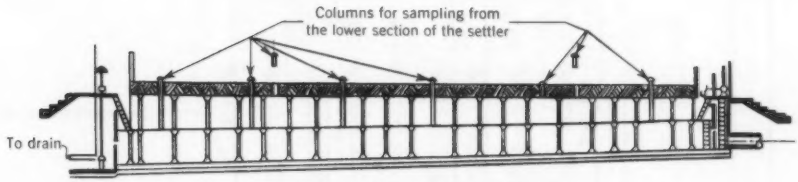
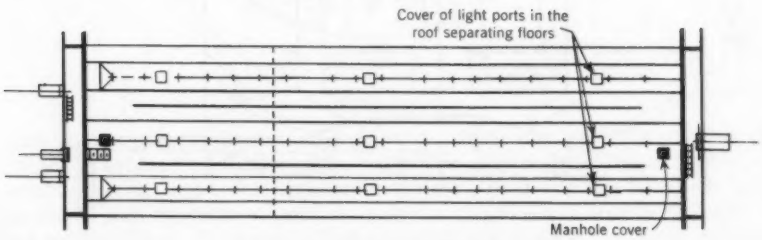


FIG. 1



(*a*) LONGITUDINAL SECTION



(*b*) LAYOUT OF ONE SECTION

FIG. 2

tracer test, indicated that the performance was comparable to the more conventional basins in use at the plant. A comparison of the two-floor basin with a single floor basin of the same capacity is shown in Table 2.

An upflow and contact type coagulation-filtration unit is being used at some municipal and industrial installations. Further development work is being carried out on this unit at the present time to enable its use on medium turbidity waters (over 150 mg per l). This unit will be described in more detail subsequently.

TABLE 2.—COMPARISON OF THE SINGLE AND TWIN-FLOOR SEPARATORS OVER AN AVERAGE SERVICE PERIOD OF 1.5 MONTHS
(Between Rinsing Periods)

Index	Two-Floor Unit of the Northern Water Supply Sta.	Single Floor Separator of the Stalin Water Supply Station
Volume in cu. m.	27,330	27,120
Dimensions in Layout Plan	63 x 74 meters	78 x 80 meters
Area Occupied by Settlers	4,741 sq.m.	6,250 sq.m.
Cubic Dimensions Measured Externally	32,175 cu.m.	34,000 cu.m.
Concrete Used	4,963 cu.m.	5,311 cu.m.

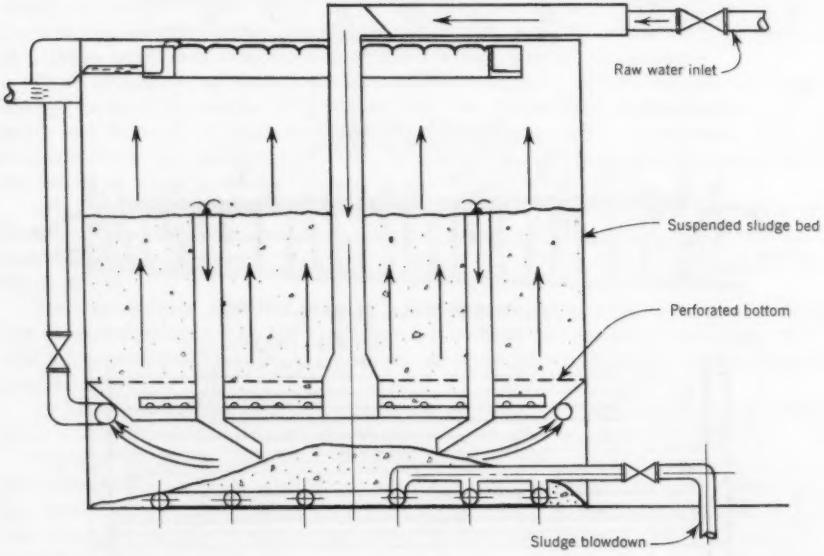


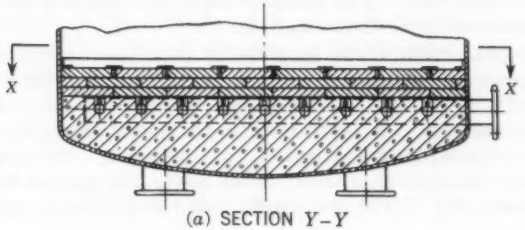
FIG. 3

Considerable development work on high rate, suspended-sludge-type clarifiers was done and a few full-sized units were installed at several municipal

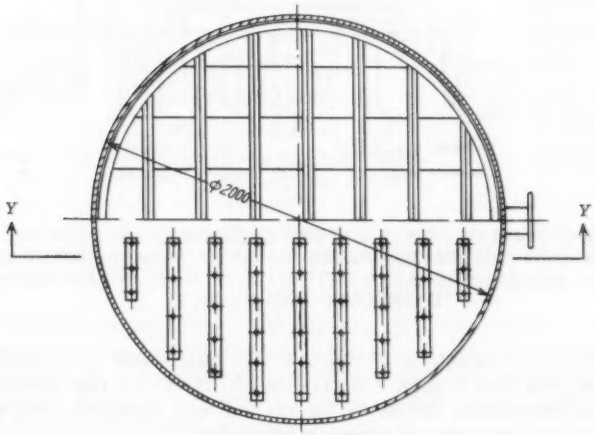
installations in 1953-54. The operating experiences with these units were so unsatisfactory that all further work along this line was abandoned until very recently. Fig. 3 is a typical design.

BREAKDOWN OF TREATMENT PRACTICES BY PROCESS AND EQUIPMENT

Ion Exchange.—The use of ion exchange for water treatment is quite extensive in Russia and the Eastern European countries, particularly for softening on the sodium cycle. The early use of natural ion exchangers (zeolites) was accompanied by investigations of other natural exchangers or adsorbents (4),



(*a*) SECTION *Y–Y*



(*b*) SECTION *X–X*

FIG. 4

but widespread use of ion exchange for water softening was dependent on development of synthetic cation exchangers. The sulfonated carbon type of cation exchanger was apparently put into industrial use in the late 1940's or early 1950's and is still the only industrial synthetic cation exchanger in general use.

Cold Sodium Cycle Softening.—The design of cold sodium cycle ion exchange units naturally parallels development of this process in the United States and Europe, although several interesting differences are noted. The later designs include upper and lower distributing systems (translated literally as "drainage plates") which are shown partially in Fig. 4. Upper distributors were of

various designs in the early units but later designs were fairly standardized with a flat plate type using standard "caps" or nozzles designated by VIT (5).

The under drainage systems utilized various distributors such as header lateral type, with quartz sub-fill, all of which was covered by the lower cover plate. It is not clear what purpose was served by the cover plate, since practice in the United States for sodium-cation exchangers places the ion exchanger directly on the sub-fill. Considerable breakage was experienced with these cover plates due to sudden pressure rise under the plate during backwash (5). Some experimental designs used porous, inert plates laid directly on a concrete fill, in which was imbedded the header-lateral distributors. The experiments claimed a 10% increase in permissible resin volume with an increase of pressure drop of less than 3% as compared with an identical exchanger with a conventional, covered under-drain system.

Sea Water Regeneration.—The location of petroleum and chemical industries and accompanying population centers in the region of the Black Sea led at an early date (1932-40), to the use of sea water for regeneration of natural zeolites. More recently, (6) tests have been made, using the waters of the Caspian Sea for regenerating sulfonated coal softeners. Because of the relatively low sodium ion content of the Caspian Sea (3,180 ppm as Na^+) as compared to the Black Sea (5,100 ppm as Na^+) and also because of the relatively

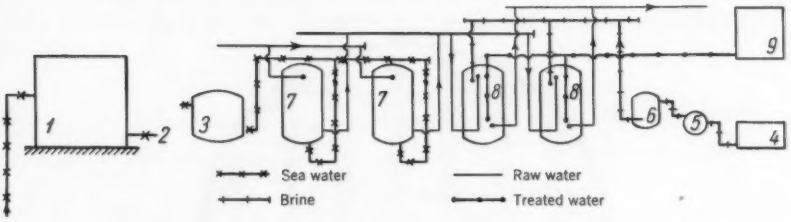


FIG. 5.—FLOW DIAGRAM OF SEA WATER REGENERATED SOFTENERS. (1: SEA WATER TANK; 3: SEA WATER FILTER; 4: BRINE TANK; 6: BRINE FILTER; 7: SEA WATER REGENERATED CATION UNITS; 8: BRINE REGENERATED "BARRIER" EXCHANGERS.)

higher molar concentration of calcium, it was impossible to obtain an effluent with hardness less than 5 ppm. For this reason the industrial softening unit at the Bakiniskii Petroleum Refinery IM Stalina was designed with a primary softening unit regenerated with Caspian Sea water.

A secondary or "barrier" unit, regenerated with commercial salt, was used to remove the residual hardness. Fig. 5 shows a flow diagram of this plant. The overall reduction in use of commercial salt was on the order of 20 to 1, although reduced capacity of the ion exchanger regenerated with Caspian Sea water (less than 1/3 the normal capacity) resulted in a considerably larger initial expenditure for equipment.

Other Ion Exchange Processes.—The use of ion exchangers for hydrogen-sodium blend systems and complete demineralization was limited until very recently by the lack of good, stable, low cost synthetic ion exchangers. In 1956 one or two demineralizing plants were installed for treating makeup to high pressure boilers and some experimental work was done concerning the potability of demineralized water using the synthetic resins available. Table 3 indicates the types of synthetic resins available in the U.S.S.R. at the end of 1957.

Cold Processing Softening.—The classical method of softening and alkalinity reduction using lime, supplemented with soda ash, caustic soda or phosphates, has been well-known and extensively used in Russia and in Eastern European countries for many years. The continuous-flow type of equipment, with external sludge recirculation, is widely used in Russian industrial plants and in some cases in municipal plants, although relatively long detentions require extensive equipment outlays. This is similar to German and European practices in the late 1940's and early 1950's.

There has been no widespread usage of the "high rate" upflow type of softening and clarification equipment such as sludge blanket or sludge (slurry) recirculation designs which have been in use in the United States for many years. An early design installed at several locations in the U.S.S.R. is shown in Fig. 3. This unit operated on the principal of a suspended sludge bed supported on a perforated floor. This design appears to be comparable to the suspended

TABLE 3

Ion Exchanger	Type	Availability
Sulfonated Coal	Cation (Low Capacity)	Standard Industrial
Wofatite R	Cation (Low Capacity)	Small Quantities
EDE-10P	Anion (Intermediate Basicity)	Standard Industrial
PEK	Anion (Strong Base)	Experimental
AV-15	Anion (Strong Base)	Experimental
AV-16	Anion (Strong Base)	Experimental
AV-17	Anion (Strong Base)	Experimental
Wofatite L-160	Anion (Weak Base with Some Capacity for Silica Removal)	Imported
AN-2F	Anion (Weak Base)	Standard Industrial

sludge-blanket units used in the United States. However, there is no indication of any original work in Russia, Europe, or elsewhere on the concepts and ideas of solids-contact in clarification, coagulation and lime softening through the use of sludge or slurry recirculation by internal pumping as used so effectively in units reduced in the United States.

Later Russian designs incorporated a small clear-water collector in the bottom section to force sludge from the suspended bed into the bottom sludge-removal section. Numerous operating difficulties later caused this design to be abandoned. Apparently this was due to plugging of the perforated bottom and is better described by the authors of a Czech survey (7):

> "In actual service at the plant the perforated bottom proved to be a source of much difficulty which adversely affected the operation of the entire equipment. . . . an indication of the extent of these deleterious phenomena has been given by Candidate Varnelli from the Vodgeo Institute when checking the performance of the clarifying units of the Sludovski and Kibkshev Water Supply Systems in Gorkii. There were instances of the

obstruction of the perforated bottom with packed sediment to the extent of 40-50% of the area of the bottom. The layer of deposit on the perforated bottom reached as high as 1-1.2 meters. In addition, considerable obstruction was observed in the circulating area under the bottom."

Since the bulk of the surface waters are relatively high in hardness and alkalinity, and also because many industries and utilities purchase water from municipal treatment plants, partial softening and silica reduction is practiced in many municipal treatment plants.

V. M. Kyvatkoskii and L. M. Zhivilova (2) have done considerable experimental work in determining control limits, predicting dosages, and, in general, investigating the effects of pH, coagulants, and magnesia on softening and silica reduction.

Although similar investigations have been conducted in the United States, one interesting aspect of their work was the reduction of silica from 10 ppm to 15 ppm to less than 1 ppm, using raw waters with varying alkalinity and hardness. Several conditions were established as being necessary for complete silica removal:

1. Control of pH in the range of 10.1 to 10.3;
2. Sufficient magnesia present (in the ratio of 15 ppm of Mg to each ppm of silica removed), either by precipitation from the raw water or by addition of Mg ion; and
3. Sufficient contact with previously formed and recirculated magnesium sludge.

Several difficulties are noted in applying the correct treatment:

a. Control of the process by means of acidimetric titration was not reliable, since the actual OH^- concentration as determined from the pH measurements differed considerably from the OH^- concentration as determined by titration.

b. With certain low alkalinity raw waters, it was not possible to maintain the optimum pH for silica reduction without predicing an unstable effluent (that is, a decrease in the alkalinity of the treated water of 5 ppm to 25 ppm in passing through marble filters) although in many cases, increasing the coagulant dosage by 40 ppm to 60 ppm corrected the instability of the treated water to less than 5 ppm.

The Combination Clarifier-Filter.—A simplified, condensed method of clarification and filtration was developed and tested in 1953-54 at several municipal and industrial sites (Moscow, Leningrad, Cheliabinsk, and Gorkii). These tests indicated that with certain water supplies, the clarified and filtered effluent from the "contact clarifier" was equal to the effluent from conventional sedimentation and filtration equipment. The average values of water quality apparently satisfied the requirements of GOST (All-Union States Standard Committee).

The design of the "contact clarifier" is shown in Fig. 6. The raw water is treated with coagulant directly ahead of the inlet distributor. The water is distributed by header-lateral type pipe distributors at the bottom of the sand and gravel bed and flows upward through the filter media. Clarified and filtered water is collected at the top of the clarifier. All of the design data are not available (8) but from the area and flow figures given for a contact clari-

fier in operation at the Yuzhnoya Water Station of Leningrad, the writers calculate an upward flow rate of approximately 7 gpm per sq ft. of area. The graded filtered media depth is about 8 ft 6 in. and the total detention time is about 12 to 16 min.

It is stated that the equipment is not suitable, at present, for operation on water supplies with turbidities in excess of 150 mg per l. Early designs also encountered problems with shifting of the various grades of filter media, which were corrected by better distributor arrangements. Table 4 is a comparison of the contact clarifier with conventional equipment operating on the same water supply.

Unfortunately, no information concerning length of run between cleanings, methods used for cleaning sludge from the filter media, or frequency of replacement of filter media is available (8), but information from the Czech paper on high rate clarifiers (9) indicates that during the spring floods of 1956,

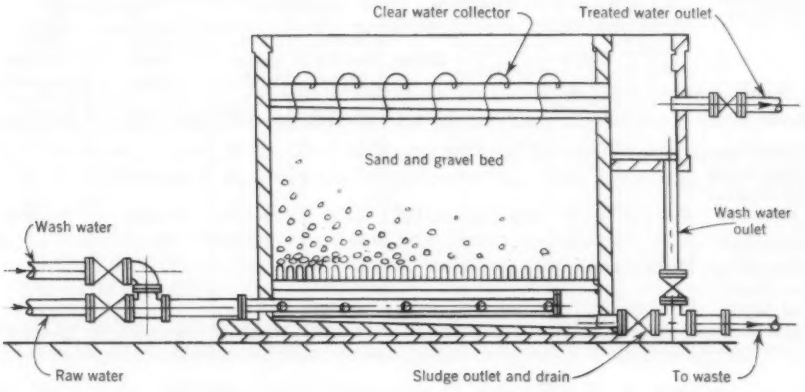


FIG. 6

the several contact clarifiers installed in the U.S.S.R. were unable to cope with the high turbidities encountered. Wash-water requirements were 28% or more of the total water treated and runs between washings were shortened to the extent that the systems often became inoperable.

Instrumentation and Automation.—In recent years, the amount and quality of instrumentation in water treatment plants in the U.S.S.R. has increased. This is particularly noticeable in the rather complex demineralization plants constructed for makeup to high-pressure boilers (10) (11) (12). Flow-metering instruments are provided in the main stream where required, and in the regenerant lines as well. Quality control instruments (conductivity, pH) are also used to check effluent and regenerant conditions, although the continuously monitoring type of instrument with automatic control and alarm contacts was not in evidence. No silicometers or other continuous analyzing equipment have yet been developed.

Hydraulically operated valves, manually controlled, are used in many of the larger plants, but the completely automatic ion-exchange units which are common in the United States are not found, indicated, or described anywhere in the literature or by visitors to the U.S.S.R.

Much of the instrumentation is fairly standardized and is manufactured and supplied by a particular industrial group. As an example, the water meters used for the demineralizing plant at one of the Mosenergo Stations were described as "Salt Meters TsLEM," a type widely used in the chemical industry.

The pH meter was developed by the Central Research Institute for Boilers and Turbines, designated TsKTI. There was no evidence in the literature of the wide use of flow, conductivity and pH monitoring of water-treatment plants, although the newer thermal electric stations (boilers and turbines) are very well instrumented and controlled.

TABLE 4.—OUTCOME OF PURIFICATION WITH CONTACT CLARIFIER

Type of Water	Cloudiness, Coloration Indices Mean Value Per Cycle				Bacteriological Data			
	Mean Conditions		Flood Conditions		Mean Conditions		Flood Conditions	
	Cloud MG/L	Color	Cloud MG/L	Color	Gastr. Bacter. Index	Bact. Count Per ML	Gastr. Bact. Index	Bact. Count Per ML
River Water	6.4-9.8	22-72°	16.7-215	42-115°	30-3000	200-2000	200-200,000	150-12,000
Water Issuing from the Contact Clarifier	0.4-1.5	9-18°	0.5-1.5	12-18°	4-72	700-800	7-2000	7-3000
Water Issuing from the Standard Apparatus	1	11°	0.6-1.8	8-19°	4-16	10-590	4-1400	6-4000

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DIFFUSION IN A SECTIONALLY HOMOGENEOUS ESTUARY

By Richard Kent[1]

SYNOPSIS

Turbulent diffusion in a well-mixed tidal estuary is considered. A practicable theory is presented from which quantitative predictions of mean pollutant-concentration distributions may be obtained for an estuary characterized by a steady-state salinity and velocity fields which are considered to vary only in the axial direction.

INTRODUCTION

This work concerns turbulent dispersion of a transient-state pollutant distribution within a tidal estuary. The purpose of the investigation is to provide a practicable theory from which quantitative predictions of mean pollutant-concentration distributions may be obtained for an estuary characterized by steady-state salinity and velocity fields which are considered to vary only in the axial direction.

From the practical standpoint, solutions to problems concerning tidal flushing in estuaries and other inshore waterways are of considerable importance. In particular, a means of predicting the quantities of public and industrial wastes that can be removed and neutralized by natural circulatory systems is desirable. An additional and no less significant application of flushing solutions concerns problems of civil defense.

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[1] Pres., Natl. Marine Consultants, Inc., Santa Barbara, Calif.

For some years, owing to the lack of an adequate turbulence theory, many investigators in fluid mechanics and related fields have assumed the phenomenon of turbulent transport to be analagous with that of molecular conduction. As a consequence of this assumption, the differential equations of turbulent diffusion appear as variations of the familiar classical form

$$\frac{\partial B}{\partial t} = \nabla (K \nabla B) \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which B is a concentration measure of the diffusant, K denotes the kinematic coefficient of eddy diffusion, and t is time. When K is a constant, Eq. 1 reduces to Fick's second law of diffusion.

The present work shows that, theoretically, the transient dispersion of pollutant in a well-mixed estuary may be described by a one-dimensional equation, the solution of which is obtained by numerical means and uses an eddy diffusivity for pollutant which is formulated as the product of the eddy diffusivity for salt and the ratio of the extents, within the estuary, of the pollutant and salt distributions.

For evaluation, the flushing theory is applied to diffusion tests conducted in scaled model estuaries by the Corps of Engineers, United States Department of the Army. Comparisons of the theoretical predictions with the observed data show the method to be practicable and capable of providing quantitative flushing information.

LITERATURE REVIEW

During the decade from 1950 through 1959 many important advances towards the understanding of estuarine dynamics have been made. However, although the intimately associated flushing problem has been recognized for some time by engineers and physical oceanographers alike, the amount of effort devoted to its solution on theoretical or empirical grounds has been disproportionately limited.

J. P. Tully studied(1)[2] the flushing characteristics of Alberni Harbour which is part of a vertically stratified, fiord-type estuary.

B. H. Ketchum developed(2) a semi-empirical technique which permits calculation of the average steady-state distributions of fresh and salt waters in a vertically and laterally homogeneous estuary.[3] A. B. Arons and H. Stommel placed Ketchum's fundamental ideas on a formal mathematical basis.(3) H. Stommel has proposed(4) a means of computing steady-state pollutant distributions in a sectionally homogeneous estuary.

In addition to the semi-empirical and theoretical research by Tully, Ketchum, Arons and Stommel, extensive studies of tidal flushing have been made in model estuaries by the Corps of Engineers.(5,6,7,8) These particular investigations will be considered in the following section.

FLUSHING INVESTIGATIONS IN MODEL ESTUARIES

Experimental Models.—Extensive studies of diffusion have been conducted by the Corps of Engineers in two model estuaries located at the Waterways

[2] Numerals in parentheses, thus (1), refer to corresponding items in the Bibliography.
[3] Hereafter referred to as a sectionally homogeneous estuary.

Experiment Station, Vicksburg, Miss., the results of which were made available for use in this work.

The first model in which diffusion was investigated is an elegant scaled reproduction in plaster and concrete of the entire estuary of Delaware Bay and River from Capes May and Henlopen to Trenton, N. J. This model is constructed to a horizontal scale of 1:1000 and vertical scale of 1:1000. These scales fix the ratios of velocity, time, slope, river discharge and channel volume. The mass field ratio is unity. One prototype tidal cycle of 12 hr and 25 min is reproduced in the model in 7.45 min. Appurtenances used in the flushing studies include tide generators, tide recorders, tide gauges, inflow weirs, current meters, salinity meters, chemical titration units and automatic water-sampling units. A plan view of the estuarine system is shown in Fig. 1.

The Delaware model estuary, like most free-surface hydrodynamic models, was constructed according to Froude scale. The hydrostatic verification was accomplished by empirical roughening. Average values of the maximum tidal-velocity amplitudes are 3 fps. Satisfactory reproduction of the prototype mass field of salt seemed to be a consequence of the hydraulic and kinematic adjustments. Of particular importance is the fact that the nature of the salinity field in the Delaware estuary is not a well-defined salt wedge. Over the greater part of this system, within the limits of salinity intrusion, the salt concentrations are essentially the same vertically and laterally and there is almost no difference in the phase of the current with depth. Such an estuary is well suited to one-dimensional flushing analysis. Representative data demonstrating the agreement between model and prototype salinities can be found in Fig. 2.

The second model in which flushing tests were conducted was designed for basic research. It is a lucite flume 327 ft long, 0.75 ft wide, and 1.5 ft deep. The flume is connected to a basin of substantial size, thus making for easy control of tidal amplitudes and periods.

Investigational Procedure.—The flushing experiments concerned transient diffusion of a conservative pollutant along the axes of the model estuaries. In each flushing test a known mass of methylene-blue dye was introduced into the model at a particular location. Immediately, intense tidal mixing caused the dye to diffuse in all directions. After a short period of two or three tidal cycles, the diffusant assumed and maintained a distribution which was somewhat bell-shaped along the axis of the estuary and uniform sectionally. The disposition of the dye-mass was obtained from samples of the estuarine water withdrawn at selected time and space intervals. The samples were examined spectrophotometrically and the concentrations of methylene-blue dye determined. Owing to the sectional uniformity of dye concentrations, the observed transient diffusion phenomenon was essentially one-dimensional in nature.

In all tests the dye was introduced into the model estuary at tidal high-water slack. The motion within the estuary of the main body of dye was a downstream-upstream tidal oscillation upon which the net velocity field imposed a seaward translation. In each experiment the mass and momentum fields were kept in steady state.

Several specifically designed tests were carried out in the model estuaries varying the following conditions:

a. Release point of dye;
b. Magnitude of river discharge; and
c. Amplitude of tide.

FIG. 1.—LOCATION MAP OF THE DELAWARE ESTUARY.

In each flushing test the spatial distribution of the dye rapidly attained a one-dimensional aspect. The decay rates of peak dye concentration were much greater in the first two tidal cycles than for later times. This abrupt change may be seen in Fig. 3. Here the decay rate up to cycle 2 and the decay rate beyond cycle 2 are approximated by straight lines A and B. D. W. Pritchard has shown(9) that the relationship between peak dye concentration, in parts per million, and time, in tidal cycles, may be approximated by

$$B_{peak} = a_i e^{-b_i t} + a_f e^{-b_f t} \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which B is dye concentration and the remaining parameters are given in Fig. 3. The extreme decay rates during the first two or three tidal cycles

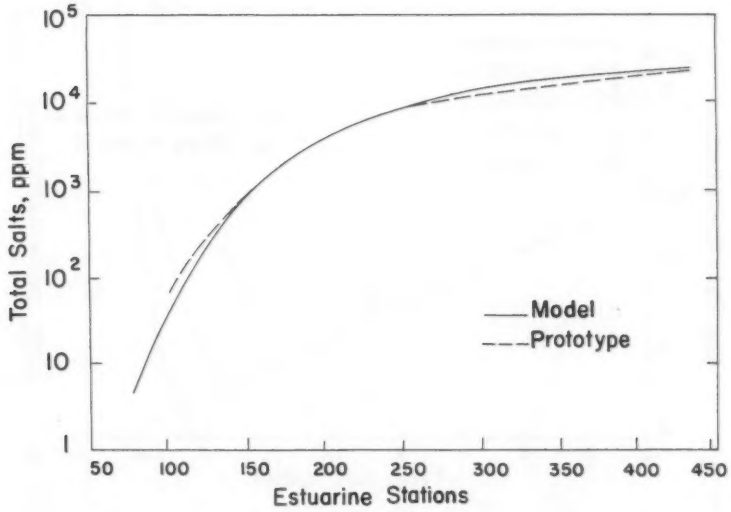

FIG. 2.—RELATIONSHIP BETWEEN SALINITY AND STATION

correspond to the dominance of the first term of Eq. 2 when $0 < t \leq 3$. For times greater than two or three tidal cycles, the peak decay rate of the dye distribution is much less extreme and the second term in Eq. 2 is dominant. Thus, it would appear that the processes governing the diffusion of dye during the initial stages differ from those effective at later times.[4]

Experimental Results.—

Delaware Model.—

a. A distinct relation existed between the decay rates of peak dye concentration and the location of dye release. The nature of their connection may be seen in Fig. 4. These graphs indicate the peak decay rates in the early and middle stages to be directly related to the release locations.

b. There was evidence of a connection between peak decay rate and the magnitude of the river discharge.

[4] The Flushing analysis developed in this paper will not consider the very early stage of transient diffusion (first three tidal cycles).

c. Although some relationship between tidal amplitude and peak decay rate was expected, no definite connection seemed evident.

d. A noticeable feature of most tests was that the transient distribution of dye in the estuary was skewed downstream. A very important flushing characteristic of every test, one which will serve as a significant criterion in subsequent analysis, was the presence of the flushing anomaly, defined as

$$\int_{t_0}^{t_i} \bar{v}\, dt - S_p > 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

where $\bar{v}$ (x) is the mean axial component of the net velocity, $\int_{t_0}^{t_i} \bar{v}\, dt$ is the distance along the estuary a particle coinciding with the pollutant peak at time t_o

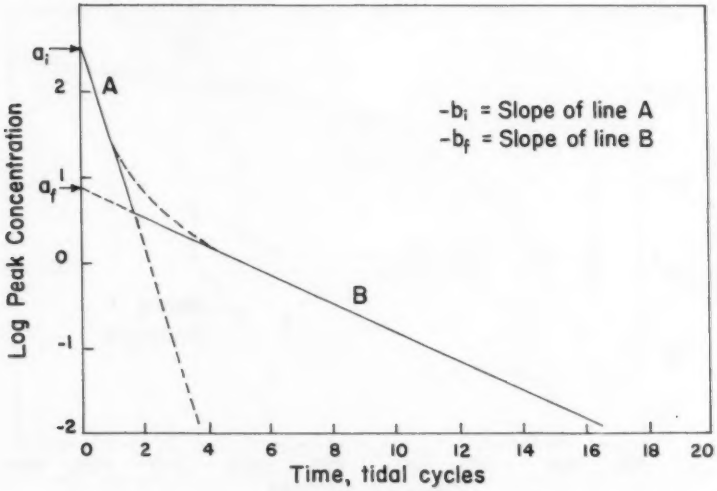

FIG. 3.—RELATIONSHIP BETWEEN PEAK DYE CONCENTRATION AND TIME.

will be advected by the mean velocity field in the interval $t_i - t_o$, and S_p is the distance separating the positions of the pollutant peak at t_o and t_i. The flushing anomaly is thus seen to be the discrepancy between the actual position of the pollutant peak and the position it would be expected to occupy were it transported with the mean field velocity alone. It was found that for M_p, the distance separating the two locations of the medians of the pollutant mass distributions at times t_o and t_i,

$$S_p < M_p < \int_{t_0}^{t_i} \bar{v}\, dt \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

As a convenience, the initial displacement of the dye release point from Station 00 was added to the computed values of $\int_{t_0}^{t_i} \bar{v}\, dt$, S_p, and M_p, thus identifying them at any time t_i, with the station numbers shown in Fig. 1. These relationships are shown quantitatively in Fig. 5 for a typical flushing test.

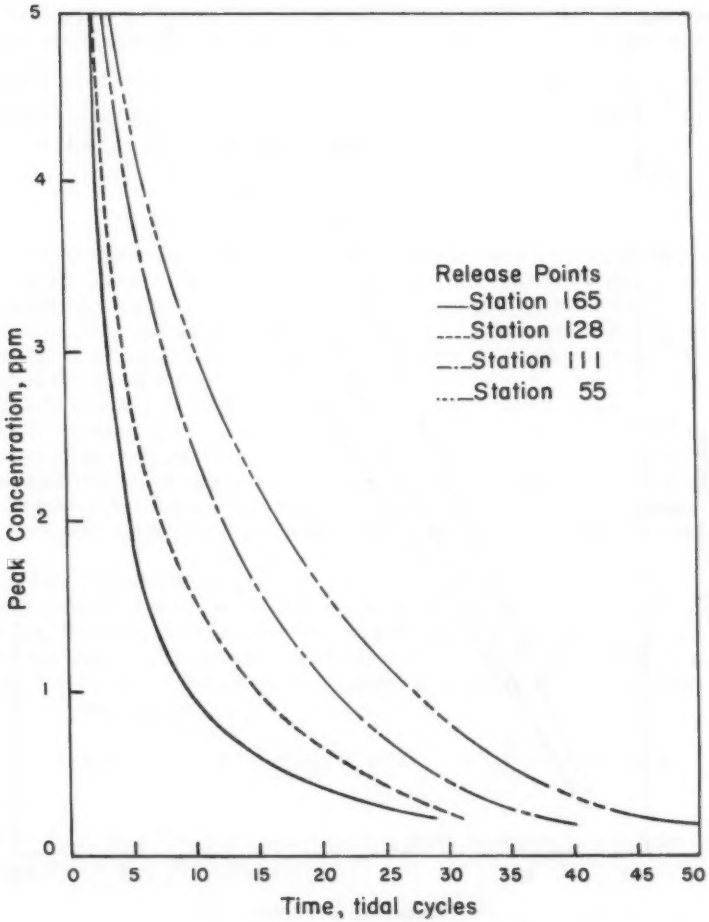

FIG. 4.—RELATIONSHIP BETWEEN DECAY RATE OF PEAK CONCENTRATION AND LOCATION OF DYE RELEASE.

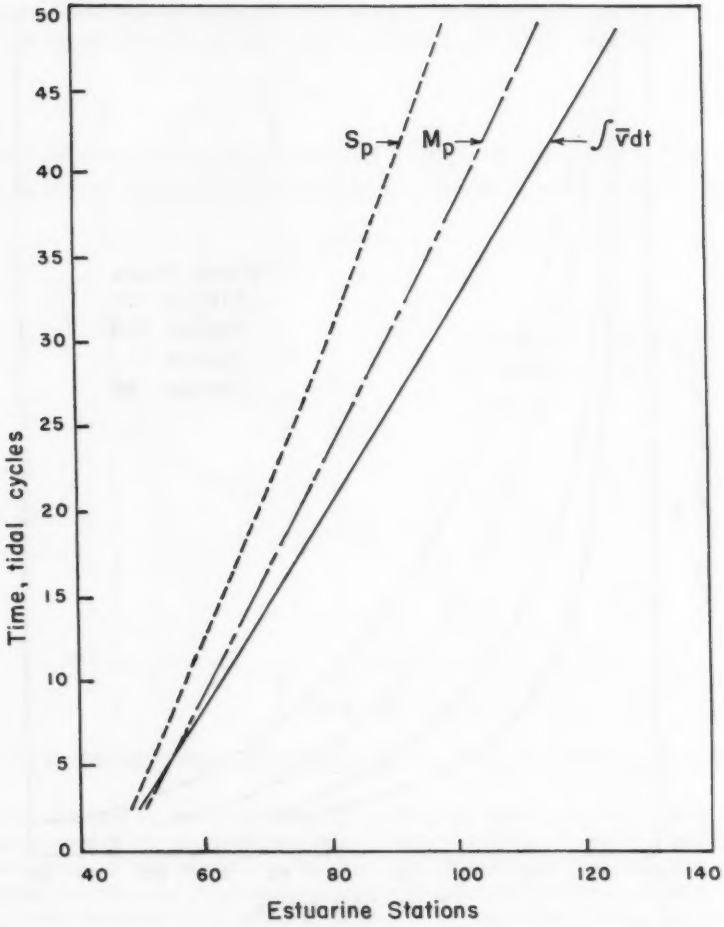

FIG. 5.—VALUES OF S_p, M_p, AND $\int \bar{v}\ dt$ VERSUS TIME FOR A TYPICAL FLUSHING TEST (TEST 9).

Lucite Model.—Although several flushing tests were conducted in the lucite flume, only three of them were amenable to a one-dimensional analysis. These three tests were performed under conditions of varying river discharge. Tidal amplitudes and the locations of dye release were the same for all tests.

The data from the flume tests also indicated a direct relationship between the magnitude of river discharge and the peak decay rate (Fig. 6), and the tendency of $\int_{t_0}^{t_i} \bar{v}\, dt$ to exceed S_p and M_p. Of considerable interest in this group of experiments was the presence of the flushing anomaly (Eq. 3) during zero river discharge.

THEORY

Equation of Estuarine Diffusion.—Pritchard has used(10) salinity structure as a basis for classifying estuaries. According to Pritchard's classification, a type-D estuary is one so intensely mixed by tidal forces that it is sectionally homogeneous. The estuarine system considered here belongs to this category. For the important assumption of gradient diffusion, turbulent transport will be confined to the axial direction of the estuary.

A type-D estuary is shown schematically in Fig. 7. A steady river flow entering at the head of the system is the only source of fresh water. At a given instant let a known mass of conservative pollutant, completely miscible with the estuarine water, be introduced into the estuary. Assume the mass of material disperses in such a way that after a brief interval its mean concentration distribution within the estuary is somewhat Gaussian axially and uniform sectionally.

Consider an elemental segment of the estuary which is formed by two cross sectional areas A and A' separated by an infinitesimal distance δx_1 (Fig. 7) and let q(x, t) denote the concentration of pollutant per unit volume of mixture.

Since the estuary is bounded laterally and vertically, the net transport of pollutant into or out of the elemental volume must be in the x_1-direction. The net transport is given by

$$v_i\, q\, A - \left\{ v_i\, q\, A + \frac{\partial}{\partial x_i} (v_i\, q\, A)\, \delta x_i \right\} = - \frac{\partial}{\partial x_i} (v_i\, q\, A)\, \delta x_i \quad \ldots . \quad (5)$$

where i = 1, 2, 3. The net transport of q must be equal to the time rate of change of pollutant in the unit volume

$$\frac{\partial q}{\partial t} A\, \delta x_i = - \frac{\partial}{\partial x_i} (v_i\, q\, A)\, \delta x_i \quad \ldots \ldots \ldots \ldots \quad (6)$$

where i = 1, 2, 3. Eq. 6 is the conservation equation for the pollutant.

Since the estuary is sectionally homogeneous in q, it is clear that

$$\frac{\partial q}{\partial x_2} = \frac{\partial q}{\partial x_3} = 0 \quad \ldots \ldots \ldots \ldots \ldots \quad (7)$$

and

$$\frac{\partial q}{\partial x_i} \equiv \frac{\partial q}{\partial x_1} \quad \ldots \ldots \ldots \ldots \ldots \quad (8)$$

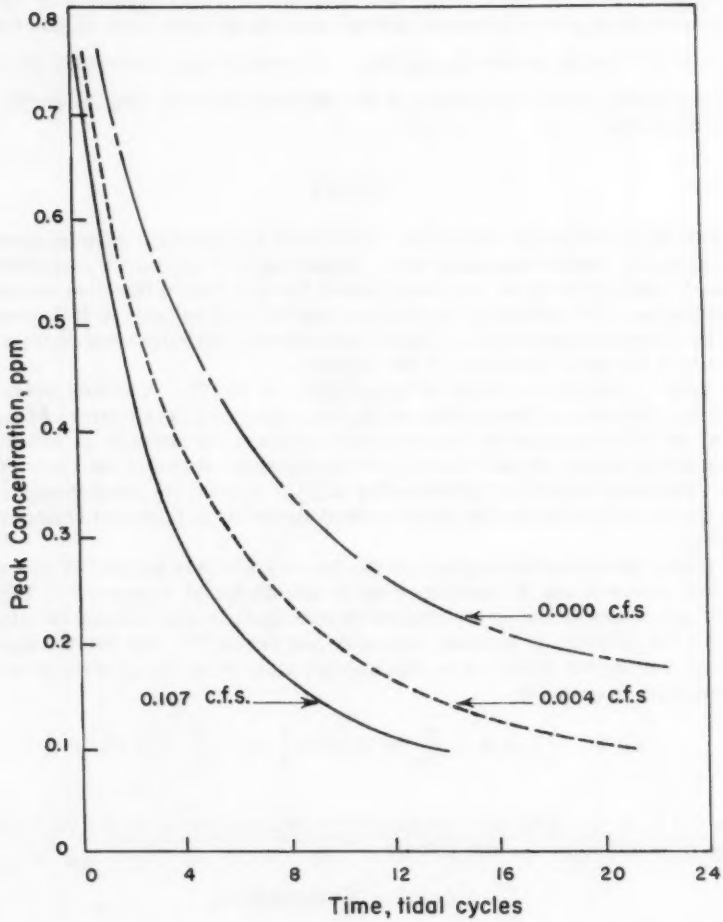

FIG. 6.—RELATIONSHIP BETWEEN DECAY RATE OF PEAK CONCENTRATION AND MAGNITUDE OF RIVER DISCHARGE FOR THREE FLUME TESTS.

Furthermore, if

$$v_2 = v_3 = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

and if v_1 is the cross-sectional average of the instantaneous axial velocities, it follows that

$$\frac{\partial v_1}{\partial x_2} = \frac{\partial v_1}{\partial x_3} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

With Eqs. 8, 9, and 10, Eq. 6 may be written as

$$\frac{\partial q}{\partial t} = - \frac{1}{A} \frac{\partial}{\partial x_i} (v_1 \, q \, A) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

Since Eq. 11 is essentially one-dimensional, subscripts on v and x are no longer required.

Let v and q, the axial velocity and the concentration of pollutant, respectively, be measured at a point in the estuary. The mean axial velocity is defined at a fixed point and at a time t_o by the relation

$$\bar{v} = \frac{1}{T} \int_{t_0-(T/2)}^{t_0+(T/2)} v \, dt \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (12)$$

where T is an arbitrary time interval called the period of sampling. The mean pollutant concentration is defined similarly by

$$\bar{q} = \frac{1}{T} \int_{t_0-(T/2)}^{t_0+(T/2)} q \, dt \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (13)$$

A fluctuation in axial velocity or pollutant concentration may be defined as the difference between the instantaneous value and mean value; thus

$$v' = v - \bar{v} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14a)$$

$$q' = q - \bar{q} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (14b)$$

Assume now that $\bar{v}$ and $\bar{q}$ are constant or, at most, slowly varying functions of t during the interval T. If the fluctuations are random,

$$v' = \frac{1}{T} \int_{t_0-T/2}^{t_0+T/2} (v - \bar{v}) \, dt = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (15)$$

and

$$\bar{q} = \frac{1}{T} \int_{t_0-(T/2)}^{t_0+(T/2)} (q - \bar{q})\, dt = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Upon substituting Eqs. 14 and 15 into Eq. 11 and taking a time average, indicated by angular brackets, there results

$$\frac{\partial \bar{q}}{\partial t} = -\frac{1}{A}\frac{\partial}{\partial x}(\bar{v}\,\bar{q}\,A) - \frac{1}{A}\frac{\partial}{\partial x}(A < v'\,q' >) \quad \ldots\ldots\ldots (17)$$

assuming that

$$\left\langle \frac{\partial v'}{\partial t} \right\rangle = \left\langle \frac{\partial q'}{\partial t} \right\rangle = < q' > = < v' > = 0 \quad \ldots\ldots\ldots (18)$$

and

$$< v'\,\bar{q} > = < \bar{v}\,q' > = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

It cannot be asserted, a priori, that the mean value of the products of the fluctuating quantities $<v'q'>$ is or is not equal to zero.

It is customary to express the eddy fluxes of property in terms of a coefficient of eddy diffusion and a mean gradient.

Accordingly, let the coefficient of eddy diffusion for pollutant be defined by

$$K_q = -\frac{< v'\,q' >}{\partial \bar{q}/\partial x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (20)$$

Substitution of Eq. 20 into Eq. 17 gives

$$\frac{\partial \bar{q}}{\partial t} = -\frac{1}{A}\frac{\partial}{\partial x}(\bar{v}\,\bar{q}\,A) + \frac{1}{A}\frac{\partial}{\partial x}\left(A\,K_q\,\frac{\partial \bar{q}}{\partial x}\right) \quad \ldots\ldots\ldots (21)$$

Since the river discharge, R, is independent of time,

$$R = \bar{v}\,A = \text{a constant} \quad \ldots\ldots\ldots\ldots\ldots\ldots (22)$$

Substitution of Eq. 22 into Eq. 21 yields the desired diffusion equation for a sectionally homogeneous estuary

$$\frac{\partial \bar{q}}{\partial t} = -\bar{v}\,\frac{\partial \bar{q}}{\partial x} + \frac{1}{A}\frac{\partial}{\partial x}\left(A\,K_q\,\frac{\partial \bar{q}}{\partial x}\right) \quad \ldots\ldots\ldots\ldots (23)$$

It is worth noting that Eq. 17 is a satisfactory description of turbulent diffusion in a sectionally homogeneous estuary, provided the conditions expressed by Eqs. 18 and 19 are fulfilled. The applicability of Eq. 23 to problems in estuarine diffusion depends on how well the unknown turbulence mechanism is incorporated into the formulation of the coefficient of eddy diffusion.

Assume the validity of Eq. 23 and consider its exact solution:

Exact Solution of Eq. 23.—Let a known mass, S, of pollutant be instantaneously generated at a given point and allowed to diffuse along the axis of an infinitely long, sectionally homogeneous estuary. Assume sectional homogeneity of pollutant concentrations.

Eq. 23 may be expanded and written as

$$\Phi \frac{\partial^2 q}{\partial x^2} + \Psi \frac{\partial q}{\partial x} + \theta \frac{\partial q}{\partial x} = \frac{\partial q}{\partial t} \quad \ldots\ldots\ldots\ldots (24)$$

where

$$\Phi \equiv K_q, \quad \Psi \equiv \left(-v + \frac{\partial K_q}{\partial x}\right), \quad \text{and} \quad \theta \equiv \frac{K_q}{A} \frac{\partial A}{\partial x}$$

The general solution, q(x, t), of Eq. 24 must be chosen so that it satisfies the following boundary conditions:

$$q(x_o,t) = \infty \qquad \text{when } t = 0$$

$$q(x,t) = 0 \qquad \text{when } t = \infty$$

and

$$\int_{-\infty}^{\infty} q(x_i,t)\, A\, dx = S$$

The last condition conserves the pollutant.

To solve Eq. 24, the method of separation of variables is used. Assume that the desired solution, q(x, t) may be expressed as the product of two independent functions; that is

$$q(x,t) \equiv X(x)\ T(t) \quad \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

Substitution of Eq. 25 into Eq. 24 yields

$$\Phi \frac{X''(x)}{X(x)} + (\Psi + \theta) \frac{X'(x)}{X(x)} \equiv \frac{T'(t)}{T(t)} \quad \ldots\ldots\ldots\ldots (26)$$

Letting

$$T'(t) = -\beta^2\, T(t) \quad \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

we have as the solution,

$$\Phi\ X''(x) + (\Psi + \beta)\, X'(x) = -\beta^2\, X(x) \quad \ldots\ldots\ldots\ldots (28)$$

It is evident that a general solution to Eq. 23 must satisfy Eqs. 27 and 28 and the boundary conditions. The nature of the solution is dependent on the way Φ, ψ, and θ vary in the estuary.

For purposes of illustration, consider solutions of Eq. 23 for three different estuaries:

Estuary a.—Let the estuary be of uniform cross section, A, constant net velocity, v, and constant eddy diffusivity, K_q, whence Φ, ψ, and θ are constants. The general solution of Eq. 23 which satisfies Eqs. 27 and 28 and the boundary conditions is easily obtained and is

$$q(x,t) = \frac{S}{2\sqrt{\pi\ K_q\, t}} \exp\left[-\frac{(x - v\,t)^2}{4\ K_q\, t}\right] \quad \ldots\ldots\ldots\ldots (29)$$

Estuary b.—Let the estuary be of uniform cross section and constant net velocity, and let the eddy diffusivity be a linear function of x; then $\phi = a_1 x$, $\theta = a_2 x$, ψ = a constant and a_1 and a_2 are constants.

In this case, the particular solution of Eq. 23 is

$$q(x,t) \sim e^{-\beta t} x^{\nu} [A J_{\nu} + B Y_{\nu}] \quad \ldots\ldots\ldots\ldots (30)$$

where υ is the order of Bessel's equation, A and B are arbitrary constants, and J_{υ} and Y_{υ} are Bessel's functions of the first and second kinds.

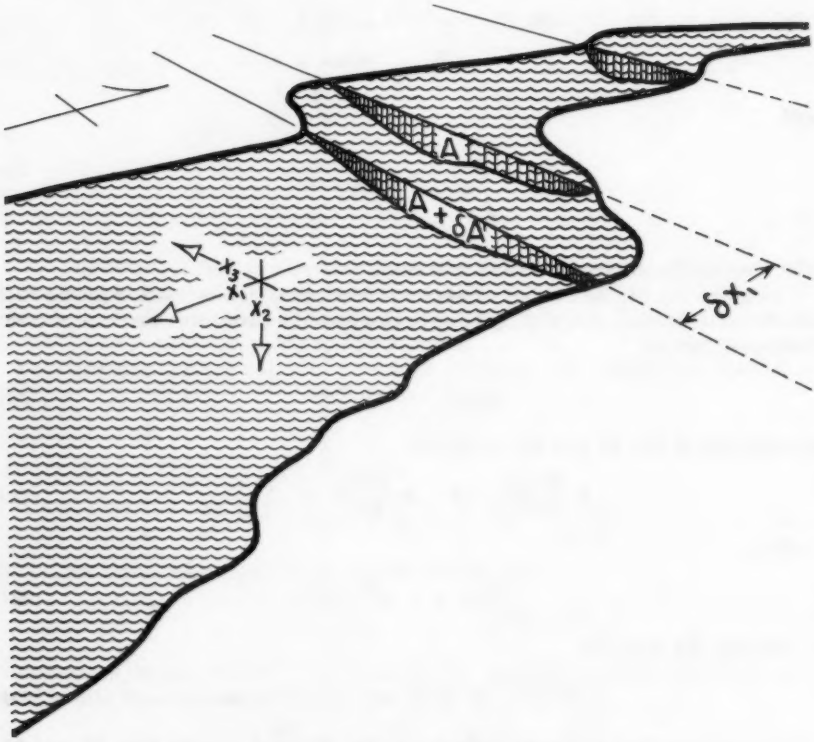

FIG. 7.—SCHEMATIC ESTUARY

Estuary c.—Let v, A, and K_q be linear functions of x. Then $\phi = \alpha x$, $\psi = \gamma x$, and $\theta = \eta x$, in which α, γ, and η are constants. The corresponding particular solution of Eq. 23, obtained from Eqs. 27 and 28 is

$$q(x,t) \sim e^{-\gamma^2/t} x^{-\alpha/\eta} e^{-x/\eta} W(p,q.r) \quad \ldots\ldots\ldots (31)$$

where W(p, q, r) is the Whittaker function in which

$$p \equiv \frac{2\gamma\eta - \alpha(\eta - 1)}{2\eta}, \quad q \equiv \frac{\alpha - \eta}{2\eta}, \text{ and } r \equiv \frac{x}{\eta}$$

Although a particular solution of Eqs. 27 and 28 is obtainable for each of the foregoing examples, the corresponding general solution obtained from use of the boundary conditions is available only when Φ, ψ, and θ are constants. Moreover, not even particular solutions of Eqs. 27 and 28 seem to exist explicitly when the coefficients Φ, ψ, and θ are non-linear variables. Since it is almost certain that any natural estuary will be non-linear in either v or A, or in both, it is unlikely that a useful general solution to Eq. 23 will emerge. Lacking an elementary analytic solution of Eq. 23, alternative methods of solution must be considered.

Numerical Solution of Eq. 23.—In many physical problems involving linear, second order partial differential equations, the relevant equation is a specialization of the form

$$\frac{\partial}{\partial\alpha}\left(\rho\frac{\partial\phi}{\partial\alpha}\right)+\frac{\partial}{\partial\alpha}\left(\eta\frac{\partial\phi}{\partial\beta}\right)+\frac{\partial}{\partial\beta}\left(\eta\frac{\partial\phi}{\partial\alpha}\right)+\frac{\partial}{\partial\rho}\left(\eta\frac{\partial\phi}{\partial\beta}\right)+d\phi=f \quad \ldots(32)$$

where Φ is the dependent function and coefficients ρ, η, γ may be either constants or functions of the independent variables α and β. Eq. 32 is said to be self-adjoint of elliptic, hyperbolic, or parabolic type according as the discriminant, $(\eta^2 - \rho\gamma)$, is negative, positive, or zero, respectively. Parabolic differential equations are usually associated with initial-value problems. It is evident that the form of Eq. 23 is given by Eq. 32 when $(\eta^2 - \rho\gamma)$ equals zero and α and β represent space and time.

In order to obtain a numerical solution to an initial-value problem, it is often convenient to replace the parabolic differential equation by its finite difference analog. The solution to a finite difference formulation of an initial-value problem is assured, generally, when the corresponding exact solution exists.

In the following numerical analysis it is expected that a unique solution of the general form of Eq. 23 does exist but is not determinable in a useful form. Two finite-difference formulations are presented. The first formulation corresponds to the most general form of Eq. 23. The second corresponds to the special case where Eq. 23 has constant coefficients. In the second case, the general solution is determinable.

The purpose of the second formulation is to show the quantitative nature of the approximate solution with respect to the exact solution and to point out certain difficulties in the gross requirements for convergence. The corresponding numerical solution to the diffusion equation with variable coefficients is considered to be analagous in behavior.

Diffusion Equation with Variable Coefficients.—At a given time, t_o, let a known mass, S, of pollutant be instantaneously generated in and allowed to diffuse along the axis of an infinitely long, sectionally homogeneous estuary. At time $(t_o + \Delta t)$ let the concentration distribution of pollutant be known and expressible by an initial-value boundary condition.

Consider the finite-difference equation corresponding to the exact differential equation, Eq. 23, in which v, A, and K_q are positive, variable coefficients.

Let the initial-value boundary condition be given by

$$q(x,t) \sim e^{-x^n/t} \quad \ldots(33)$$

Additional boundary conditions are

$$q(x,t) = 0 \quad \ldots(34)$$

when $t = \infty$, and

$$\int_{-\infty}^{\infty} A\, q\, dx = S \quad \ldots\ldots\ldots\ldots\ldots\ldots (35)$$

where S is the total mass pollutant.

The forward finite-difference replacements of the various partial derivatives in Eq. 23 are

$$\frac{\partial q}{\partial t} = \frac{q_{x,t+\Delta t} - q_{x,t}}{\Delta t} - \frac{\Delta t}{2} \frac{\partial^2 q}{\partial t^2} \quad \ldots\ldots\ldots\ldots (36a)$$

$$\frac{\partial q}{\partial x} = \frac{q_{x+\Delta x,t} - q_{x-\Delta x,t}}{2\,\Delta x} - \frac{\overline{\Delta x}^2}{6} \frac{\partial^3 q}{\partial x^3} \quad \ldots\ldots\ldots\ldots (36b)$$

and

$$\frac{\partial^2 q}{\partial x^2} = \frac{q_{x+\Delta x,t} - q_{x-\Delta x,t} - 2\, q_{x,t}}{\overline{\Delta x}^2} - \frac{\Delta x^2}{12} \frac{\partial^4 q}{\partial x^4} \quad \ldots\ldots\ldots\ldots (36c)$$

where the space-time pattern of formulation is shown by Fig. 8. Substitution of the finite differences, Eqs. 36, into the diffusion equation, Eq. 23, with some rearrangement yields

$$q_{x,t+\Delta t} = q_{x,t}\left[1 - \frac{1}{A_x} \frac{\Delta t}{\overline{\Delta x}^2} \left\{(A\, K_q)_x + \frac{\Delta x}{2} + (A\, K_q)_x - \frac{\Delta x}{2}\right\}\right]$$

$$+ q_{x+\Delta x,t}\left[-\frac{v_x\, \Delta t}{2\,\Delta x} + \frac{(A\, K_q)_{x+(\Delta x/2)}}{A\, x} \frac{\Delta t}{\overline{\Delta x}^2}\right]$$

$$+ q_{x-\Delta x,t}\left[\frac{v_x\, \Delta t}{2\,\Delta x} + \frac{(A\, K_q)_{x-(\Delta x/2)}}{A\, x} \frac{\Delta t}{\overline{\Delta x}^2}\right] \quad \ldots\ldots\ldots (37)$$

where the error terms have been considered negligible. Eq. 23 is considered as the formal limit of difference Eq. 37 as Δx and Δt tend to zero.

The relationship expressed by Eq. 37 states that, within the framework of the approximating formulation, the magnitude of the dependent q at position x and time $(t + \Delta t)$ is related to the values of q at positions x, $(x + \Delta x)$ and $(x - \Delta x)$ at previous time t. Hence, if an initial distribution of pollutant in an estuary and numerical magnitudes of the parameters, v, A, and K_q, along the system are known, Eq. 37 offers a means of tracing the space-time disposition of the diffusing mass of pollutant.

For initial-value problems the existence of a solution to the associated finite-difference problem is assured, generally, when the exact problem possesses a solution. However, in order that the approximate solution converge to the exact solution, it is necessary that certain conditions involving the unit spacings Δx and Δt be satisfied.

A major requirement for convergence is that the coefficients in Eq. 37 be positive; that is

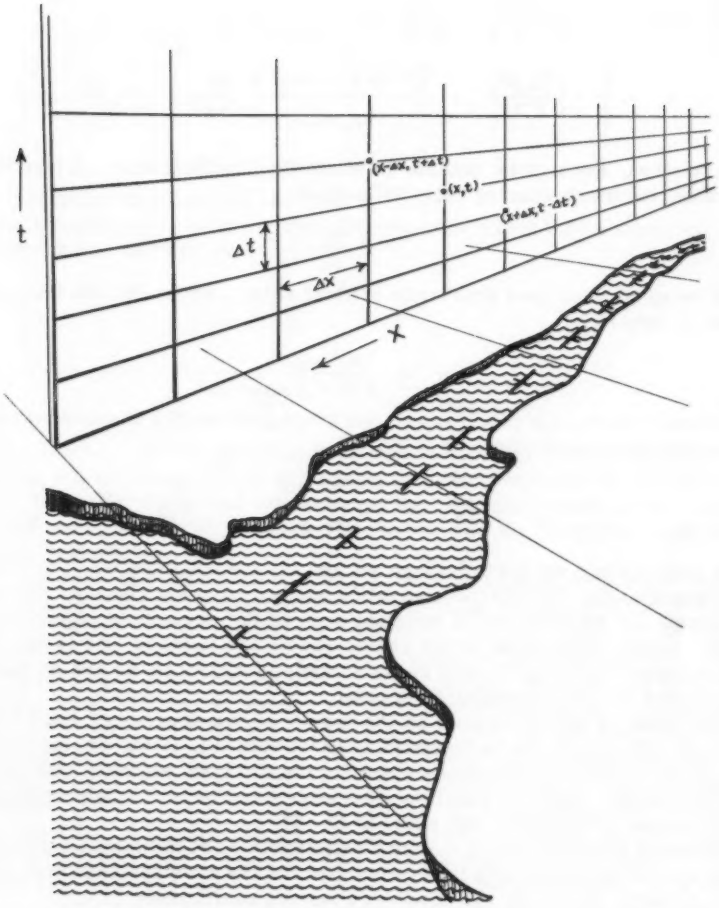

FIG. 8.—SPACE-TIME PATTERN OF THE FINITE-DIFFERENCE FORMULATION.

$$\left[1 - \frac{1}{Ax}\,\frac{\Delta t}{\overline{\Delta x}^2}\left\{(A\,K_q)_{x+(\Delta x/2)} + (A\,K_2)_{x-(\Delta x/2)}\right\}\right] \geq 0 \quad \ldots (38a)$$

and

$$\left[\pm\,\frac{v_x\,\Delta t}{2\,\Delta x} + \frac{(A\,K_q)_x \pm (\Delta x/2)}{A\,x}\,\frac{\Delta t}{\overline{\Delta x}^2}\right] \geq 0 \quad \ldots\ldots (38b)$$

Clearly then, for a given domain in which the coefficients v, A, and K_q are variable, the mesh-spacing ratio defined as

$$\gamma \equiv \frac{\Delta t}{\overline{\Delta x}^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

must be equal to or less than some critical value. From Eq. 38a this critical value is seen to be

$$\gamma \leq \frac{(A_x)_{min}}{2\,(A\,K_q)_{max}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

A second convergence requirement, one which provides for the positive character of the last two coefficients in Eq. 37, is

$$\pm\,\frac{v_x\,\Delta t}{2\,\Delta x} \leq \frac{(A\,K_q)_{min}}{(A_x)_{max}}\,\frac{\Delta t}{\overline{\Delta x}^2} \quad \ldots\ldots\ldots\ldots (41)$$

If restrictions set forth by Eqs. 40 and 41 are not satisfied, the region of determination for the difference equation will not coincide with the corresponding region for the differential equation even as the network is indefinitely refined. On the other hand, if the requirements are fulfilled, the linear finite-difference solution, Eq. 37, will be stable and convergent as Δx and Δt approach 0, provided the exact solution does exist.

The effect of net refinement on the degree of convergence of Eq. 37 cannot be shown for lack of the corresponding exact solution. However, the nature of this relationship can be demonstrated for the diffusion equation with constant coefficients for which an exact solution is determinable. It is presumed that an analogous relationship will apply to Eq. 37.

Diffusion Equation with Constant Coefficients.—Let the initial and boundary conditions be identical with those stipulated for the previous case with the exception that v, A, and K_q are positive, constant coefficients.

The pertinent differential equation is

$$\frac{\partial q}{\partial t} = -\,v\,\frac{\partial q}{\partial x} + K_q\,\frac{\partial^2 q}{\partial x^2} \quad \ldots\ldots\ldots\ldots\ldots (42)$$

with the exact solution given by Eq. 29.

The forward finite difference equation corresponding to Eq. 43 is

$$q_{x,t+\Delta t} = q_{x,t}\left[1 - 2\,K_q\,\frac{\Delta t}{\overline{\Delta x}^2}\right] + q_{x+\Delta x,t}\left[K_q\,\frac{\Delta t}{\overline{\Delta x}^2} - \frac{v\,\Delta t}{2\,\Delta x}\right] + q_{x-\Delta x,t}\left[K_q\,\frac{\Delta t}{\overline{\Delta x}^2} + \frac{v\,\Delta t}{2\,\Delta x}\right] \quad \ldots\ldots\ldots (43)$$

where the space-time pattern is shown in Fig. 8.

Since v and K_q are constants over the entire domain, the requirements for convergency and stability of Eq. 43 are

$$\frac{\Delta t}{\overline{\Delta x}^2} \leq \frac{1}{2\,K_q} \quad \ldots\ldots\ldots\ldots\ldots\ldots (44a)$$

and

$$\frac{v\,\Delta t}{2\,\Delta x} \leq K_q\,\frac{\Delta t}{\overline{\Delta x}^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (44b)$$

M. L. Juncosa and D. Young have proved(11) for the heat equation that as Δx and $\Delta t \to 0$ the linear finite-difference solution converges to the exact solution. Since Eq. 42 can be transformed into the heat equation by a simple change of variable, Eq. 43 likewise will converge to Eq. 29 as the mesh is indefinitely refined.

For the present purpose, let S, v, and K_q take on certain reasonable magnitudes. Eq. 39 has been evaluated for a given value of t to yield a distribution of q in the estuary. These values are shown in Col. 3 of Table 1. Eq. 43 has also been evaluated over the same space domain for an identical time period using three sets of intervals, Δx and Δt, which satisfy Eqs. 44. The three sets of results along with the necessary initial spatial distribution of the pollutant (boundary condition Eq. 33) are shown in Cols. 2, 4, 5, and 6 of Table 1. A comparison of Col. 3 with Cols. 4, 5, and 6 shows that as the space and time intervals, Δx and Δt, become very small, the finite-difference solution, Eq. 44, converges rapidly to the exact solution. Similarly, if requirements Eqs. 40 and 41 are satisfied, it is expected that the approximate solution, Eq. 37, will approach the corresponding exact solution as Δx and Δt become small.

In the application of Eq. 37 to determine the estuarine diffusion, it is necessary, for good results, that the magnitudes of the unit space and time intervals, Δx and Δt, be chosen so that the pollutant distribution be adequately represented. On one hand, it is necessary that very little non-linearity in the required variable occur over the unit spacings and on the other, a minimum of interpolation and extrapolation along with the preclusion of an excessive number of computations is desirable. The unit spacings, Δx and Δt, may be chosen to be so small that, so to speak, no interpolation would be required to obtain a picture of the continuous distribution. However, in the majority of geophysical problems, which usually extend over large time periods, selection of exceedingly small spacings will render the numerical method exceedingly cumbersome. The ideal procedure is to begin the numerical evaluation with as fine

a mesh as is practicable (since the initial concentration gradients will be large and variable over short distances) and to pass to larger meshes as the diffusion progresses and gradients decrease.

TABLE 1.—EFFECT OF NET REFINEMENT ON CONVERGENCE OF THE NUMERICAL SOLUTION (EQ. 43) TO THE EXACT SOLUTION (EQ. 29). ALL QUANTITIES ARE EXPRESSED IN ARBITRARY UNITS.

Position	$q(x,t_0)$	$q(x,t_{0+1})_{exact}$	$q(x,t_{0+1})^1_{num}$	$q(x,t_{0+1})^2_{num}$	$q(x,t_{0+1})^3_{num}$
1	3.24	2.91	2.81	3.05	2.90
2	4.58	3.82	3.28	3.78	3.82
3	6.06	4.80	4.01	4.73	4.83
4	7.55	5.79	4.74	5.71	5.87
5	8.82	6.57	5.46	6.60	6.67
6	9.70	7.49	6.11	7.37	7.50
7	10.00	7.93	6.64	7.90	7.98
8	9.70	8.09	7.00	8.05	8.10
9	8.82	7.93	7.10	8.06	7.98
10	7.55	7.49	6.91	7.67	7.51
11	6.06	6.57	6.43	6.88	6.66
12	4.58	5.79	5.67	6.09	5.87
13	3.24	4.80	4.75	5.07	4.82
14		3.82	3.76	4.04	3.82

1: $\Delta x = 3$, $\Delta t = 1.00$, $K_q = 4$, $v = 1$, $S = 10^2$
2: $\Delta x = 2$, $\Delta t = 0.50$ $K_q = 4$, $v = 1$, $S = 10^2$
3: $\Delta x = 1$, $\Delta t = 0.13$, $K_q = 4$, $v = 1$, $S = 10^2$

There exists no theory in difference analysis by means of which an optimum mesh may be chosen, the determination being more or less a trial and error process. Young (in unpublished notes) has shown that in some problems

$$E \sim \overline{\Delta x}^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (45)$$

where E is the difference between magnitudes of corresponding analytical and numerical solutions.

That a chosen spacing is too large may be indicated in the diffusion problem in two ways:

$$E > 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (46a)$$

and

$$\int_{-\infty}^{\infty} A\, q\, dx \neq \text{a constant} \quad \ldots\ldots\ldots\ldots\ldots (46b)$$

As Δx and Δt become small, $E \to 0$ and the conservation approximation approaches an equality. In the application of the numerical equation it is possi-

ble that total convergence and conservation will demand an impracticable mesh. In such an event, a compromise will be required; that is, in order to circumvent excessive work a relaxation in the degree of convergence must be accepted.

In addition to the selection of suitable spacings for satisfaction of requirements Eqs. 40 and 41, the practical application of Eq. 37 requires that boundary condition Eq. 35 be modified since a natural estuary will not be infinite in extent. This may be done by arbitrarily selecting some minimum value of q, say ϵ, beyond which no numerical computations are made. This curtailment in the extent of the computed distribution involves a loss in pollutant but the violation of conservation is relatively insignificant. The modified boundary condition then is given by Eq. 35 where x_a and x_b are the positions along the estuary beyond which $0 \approx q < \epsilon$.

Evaluation of Eq. 37 requires, further, that the magnitudes of v, A, and K_q along the estuary be known at all times. Clearly, v and A are directly measurable quantities, however, the eddy diffusivity is less easily obtained and will be considered in the next part.

Coefficient of Eddy Diffusion.—The analogy between the kinematic action of molecular and turbulent motions suggests that the coefficient of molecular diffusion has as its macroscopic counterpart the eddy diffusivity. However, while the coefficient of molecular diffusion is a well-defined physical property, the eddy coefficient is less amenable to precise expression.

One method that has proved useful in practical problems, is to intuit those measurable quantities that may affect the turbulent mixing and to define the eddy diffusivity through dimensional reasoning and experiment. An example of this method is the Schmidt-Sutton conjugate power-laws. Another example forwarded by Arons and Stommel(3) is

$$K(x) = 2 N \xi_0 \; T_0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (47)$$

where N is a dimensionless coefficient, ζ is tidal displacement, and v_0 is tidal current amplitude.

In this work, the concept of eddy diffusivity is not specified, a priori, but is constructed on the fundamental assumption that the distribution of any scalar property in a turbulent estuary must reflect the eddy system in the estuary. In particular, if for a given system some property can be found in steady state, then the solution of the classical diffusion equation for this property will yield explicit values for the eddy diffusivity which can be applied to any other steady state distribution on the same system. Furthermore, it is proposed that these values can be modified to apply to a non-steady distribution of property subject to the same turbulent regime. For an estuary, the salinity field will serve as the required steady-state property.

Eddy Diffusivity for Salt.—In a sectionally homogeneous estuary, the salt balance equation is

$$\frac{\partial s}{\partial t} A \; \delta x_1 = - \frac{\partial (v_i \; s \; A)}{\partial x_1} \delta x_1 \quad \ldots\ldots\ldots\ldots (48)$$

where i = 1, 2, 3, and s is salinity and molecular diffusion is assumed negligible. This may be transformed into the averaged one-dimensional equation

$$\frac{\partial s}{\partial t} = - v \frac{\partial s}{\partial x} + \frac{1}{A} \frac{\partial}{\partial x} \left(A K_s \frac{\partial s}{\partial x} \right) \quad \ldots\ldots\ldots\ldots (49)$$

where K_S, the eddy diffusivity for salt, is defined by

$$K_S = - \frac{\langle v' s' \rangle}{\partial \bar{s}/\partial x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (50)$$

Since the salt field is in steady state, Eq. 49 becomes

$$0 = - v \frac{\partial s}{\partial x} + \frac{1}{A} \frac{\partial}{\partial x}\left(A K_S \frac{\partial s}{\partial x}\right) \quad \ldots\ldots\ldots\ldots (51)$$

Integrating Eq. 51 with respect to x and multiplying the result by the cross-sectional area, A, gives

$$0 = - R s + A K_S \frac{\partial s}{\partial x}, \quad \ldots\ldots\ldots\ldots\ldots (52)$$

where R is the river flow. Eq. 52 is an expression of the net flux of salt across any section of the estuary. Solving Eq. 52 for the eddy diffusivity gives

$$K_S = \frac{R s}{A \frac{\partial s}{\partial x}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (53)$$

whose forward finite difference form is

$$K_S = \frac{R s_X 2 \Delta x}{A_X (S_{X+\Delta x} - S_{X-\Delta x})} \quad \ldots\ldots\ldots\ldots (54)$$

From Eq. 54, K_S will have a spatial variation whenever s or v vary spatially.

For most estuaries $s_X + \Delta x > s_X - \Delta x$ so that $K_S > 0$. At the head of the estuary there is no salt so that $s \to o$ and $\frac{\partial s}{\partial x} \to 0$, while at the mouth of the estuary $\frac{\partial s}{\partial x} \to 0$. Clearly, Eq. 53 is not useful in these regions.

The magnitude of K_S, as determined by Eq. 54, depends on the size of the time interval at which the observed salinities are measured. In this thesis the time interval selected is the tidal cycle. Thus, $\bar{s}(x)$ and $\bar{v}(x)$ indicate the average values at position x of the several measurements of salinity and velocity taken every tidal cycle.

Eddy Diffusivity for Pollutant.—Consider, within an estuary, a distribution, q(x, t), of pollutant described by Eq. 23 and assume the following conditions:

a. $\frac{\partial v}{\partial \epsilon} = 0$ and $\frac{\partial s}{\partial t} = 0$;

b. The terms s and q are fluid-attached properties;

c. The field of q has no inertial effect on the momentum field ($\rho_{H_2O} \equiv f_q$); and

d. The periods of the random fluctuations of properties s, v, and q are much smaller than the period of averaging (tidal cycle).

The eddy fluxes of pollutant and salt given by Eq. 20 and 50 may also be expressed as

$$\langle v' q' \rangle = r_q \sqrt{v'^2} \sqrt{\bar{q}^2} \quad \ldots\ldots\ldots\ldots (55a)$$

and

$$\langle v' s' \rangle = r_S \sqrt{\bar{v}'^2} \sqrt{v'^2}. \quad \ldots\ldots\ldots\ldots (55b)$$

respectively, where r_q and r_s are correlation coefficients and $\sqrt{\bar{v}^{12}}$, $\sqrt{\bar{q}^{12}}$, $\sqrt{\bar{s}^{12}}$ are the root mean squares of the local turbulent fluctuations in v, q, and s.

Substituting from Eqs. 20 and 50 into Eqs. 55 yields the ratio

$$\frac{K_q}{K_s} = \frac{r_q}{r_s} \frac{\sqrt{\overline{q'^2}}}{\sqrt{\overline{s'^2}}} \frac{\frac{\partial s}{\partial x}}{\frac{\partial q}{\partial x}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (56)$$

Following Prandtl, let

$$\sqrt{\overline{q'^2}} \equiv l_q \frac{\partial q}{\partial x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (57a)$$

and

$$\sqrt{\overline{s'^2}} \equiv l_s \frac{\partial s}{\partial x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (57b)$$

where l_q and l_s are lengths which must be defined. Then Eq. 56 reduces to

$$\frac{K_q}{K_s} = \gamma \frac{l_q}{l_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (58)$$

where $\gamma = r_q/r$. According to Eq. 58, the ratio of the eddy diffusivities varies directly as the product of two ratios: one, the ratio of two characteristic lengths, l_q which is a function of time and l_s which is not, and the other, the ratio of the correlation coefficients.

Under the circumstances, the ratio of the correlation coefficients, γ, might be considered a constant equal to unity. To show this, one writes γ as

$$\gamma = \frac{\langle v' q' \rangle \sqrt{\overline{s'^2}}}{\langle v' s' \rangle \sqrt{\overline{q'^2}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (59)$$

The salt field is independent of time whence

$$\langle v' s' \rangle = a_1 \sqrt{\overline{s'^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (60a)$$

in which a_1 a constant.

If γ is to be a constant, it is necessary also that

$$\langle v' q' \rangle = a_2 \sqrt{\overline{q'^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (60b)$$

in which a_2 a constant.

In Eq. 60 the constants a_1 and a_2 would appear to be functions of the steady velocity field. Since s and q are fluid-attached properties in the same turbulent regime, it is reasonable to expect the constants to be equal so that $\gamma = 1$.

The foregoing development is entirely heuristic. The justification for assuming γ equal to unity will be the success with which the final equation fits the observed phenomenon.

A connection between the eddy diffusivity and some characteristic length has been indicated by both experiment and theory. From experimental evidence and theoretical considerations, L. F. Richardson(12) and A. N. Kolmogoroff (in 1941), respectively, proposed the relationship,

$$K \sim L^{4/3} \quad \ldots\ldots\ldots\ldots\ldots\ldots (61)$$

where K, the diffusion coefficient, is a function of some scale length of the process.

Although the length L is difficult to define, in particular cases it may be conceived as representing the size of the eddies most effective in turbulent diffusion.

Experimental verification of a relation between K and a characteristic length is available from several sources. In particular, experiments by Richardson(12) and P. B. MacCready(13) have shown that Kolmogoroff's hypothesis can be used in its simplest form for meteorological problems with characteristic scales ranging from 10 to 10^8 cm. From oceanographic data, H. V. Sverdrup has compiled a table of the horizontal eddy diffusivity versus the radius of the diffusant distribution(14).

C. J. Burke(15), W. H. Munk et al.(16), and G. F. McEwen(17), also found K_h proportional to r. From observations of estuarine diffusion, Pritchard(9) showed

$$K \sim \sigma^n \quad \ldots\ldots\ldots\ldots\ldots\ldots (62)$$

where σ is the standard deviation of the diffusant distribution in space and $1.0 \leq n \leq 1.5$.

Thus, from the experimental evidence, a relationship between K and a characteristic length seems to be in order for both isotropic and anisotropic turbulent diffusion.

The characteristic lengths l_q and l_s selected for use in this work are directly proportional to the ranges of the detectable salt and pollutant distributions, respectively, within the limits of the estuary. Thus,

$$l_q = c_1 L_q \quad \ldots\ldots\ldots\ldots\ldots\ldots (63a)$$

and

$$l_s = c_1 L_s \quad \ldots\ldots\ldots\ldots\ldots\ldots (63b)$$

where c_1 is a constant of the system and L_s and L_q are the measurable extents of properties s and q. Since the salt field is in steady state, L_s has a constant value whereas L_q increases with time.

The characteristic lengths l_q and l_s may be looked upon as representing the sizes of those eddies most effective in the dispersion of pollutant and salt. These eddies, in turn, are related to the sizes of the distributions. In a similar vein, if the Eulerian correlation coefficient is given by $R_x(x, t)$, a characteristic turbulent length may be defined by the relation

$$l = \int_{x_2}^{x_1} R_x(x,t)\, dx \quad \ldots\ldots\ldots\ldots\ldots\ldots (64)$$

where the limits of integration (x_1 and x_2) define a spatial portion of the distribution outside of which $R_x(x, t) = 0$.

In view of Eq. 62, Eq. 58 reduces to the relationship

$$\frac{K_q}{K_s} = \frac{c_1 L_q}{c_1 L_s} = \frac{L_q}{L_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots (65)$$

As time increases $L_q \rightarrow L_s$ and $K_q \rightarrow K_s$.

Since $\frac{\partial s}{\partial t} \rightarrow 0$, L_s, the extent of the salt intrusion in the estuary, is a constant. The eddy diffusivity for salt, K_s, is constant at each position along the estuary and may be obtained from Eq. 54. Length L_q, corresponding to the range of the pollutant distribution, is a function of time and may be measured at a given instant. Thus, K_q, the eddy diffusivity for pollutant, may be computed as a function of space and time from Eq. 65.

NUMERICAL APPLICATION

Procedure.—Of the several flushing tests conducted in the Delaware model, two were highly suited for the application and evaluation of Eq. 37. In these two tests the region of diffusion was confined for the desired period of time (20 tidal cycles) to that portion of the estuary in which the steady-state salt and momentum fields were sectionally homogeneous.[4] This region extended from Station 100 to Station 400 (Fig. 1).

Both tests were conducted under conditions of mean river discharge (16,475 cfs) and mean tidal amplitude. The magnitudes of the space dependent eddy diffusivity for salt, K_s, were computed from Eq. 54.

In both flushing tests a concentrated solution of methylene-blue dye (pollutant) was injected at tidal high-water slack into an isolated $2\frac{1}{2}$ ft long segment of the estuary. The amount of pollutant released was adjusted so that the final concentration in the segment after a thorough mixing was 50 ppm.

The masses of pollutant were not released at the same position in the estuary. In Test I the release area was in the vicinity of Station 126.5. In Test II the release area was in the vicinity of Station 162.5.

Concentrations of pollutant were measured along the axis of the model estuary once every tidal cycle. The initial distribution of pollutant required to establish the numerical process (expressed by boundary condition Eq. 33 was that one observed at the end of the third tidal cycle.

In order to determine a spatial range of the dye distribution[5] for use in Eq. 65, a cut-off concentration value of dye was established at 0.005 ppm. Although some pollutant undoubtedly extended beyond the positions of these cut-offs, it can be shown to be incapable of altering the numerical results to any appreciable extent.

A consequence of curtailing the spatial extent of the pollutant distribution is that the conservation condition, Eqs. 33 through 35, becomes an approximation. The approximation is satisfactory during the period of analysis (first 20 tidal cycles) since the amount of pollutant cut off in the tails is a very small fraction of the entire mass.

Convergence requirements on Eq. 37, set by Eqs. 40 and 41, indicated an optimum time spacing Δt, of one-sixth of a tidal cycle. Since observations were made at intervals of one tidal cycle, comparisons of the observed and computed distributions are available only for every sixth set of computations.

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[4] Between stations 280 and 340 a weak salinity stratification developed during the turn of the current from flood to ebb.

[5] Required initially for the observed distribution and subsequently for the predicted distributions.

For the optimal Δt, convergence made it necessary to use three different values of the mesh spacing, Δx, 5,000 ft, 7,500 ft, and 10,000 ft. The increase in Δx resulted from the increasing values of K_q encountered as time passed and the pollutant distribution moved downriver.

Results.—The predicted flushing data were processed and certain of its significant features compared with the corresponding features of the observed data.

In Figs. 9 and 10 the predicted and the observed values of the peak pollutant concentrations for Test I and Test II, respectively, are shown as functions of time. Clearly, the predicted curve of the peak decay rate fits the corresponding observed curve quite well. From Figs. 9 and 10 it may be seen that in the early stages of diffusion (cycles 3 to 10) the peak concentration decay rate for Test II exceeded that of Test I. This feature, which is also shown by the observed flushing data, is consistent with the relationship shown in Fig. 4.

In Figs. 11 and 12 the predicted and the observed values of S_p and M_p for both flushing tests are shown as functions of space and time. Again, it is seen that the fit of the theoretical with the observed data is good.

The composite sets of the predicted and the observed longitudinal distributions of pollutant for cycles 5, 10, 15, and 20 of Tests I and II are presented in Figs. 13 and 14, respectively. In both tests the several fits of the predicted with the observed distributions are reasonably acceptable.

The predicted pollutant distributions for both tests show the characteristic downstream skewness of the observed distributions. From Figs. 13 and 14 all of the predicted pollutant distributions are seen to be displaced somewhat to the downstream side of the corresponding observed distributions. Also, it can be seen that the predicted magnitudes of the peak concentrations in the several distributions, with the exception of those in cycles 5 and 10 of Test II, are less than the corresponding observed values. This consistent trend resulted from an artifact of the numerical analysis wherein the pollutant was not completely conserved due to the curtailment of the distributions, and to the incomplete convergence of Eq. 37 arising from a coarse mesh.

DISCUSSION

Numerical solution of the classically based equation of estuarine diffusion in which the eddy diffusivity is a variable related to the associated salt field has provided quantitative predictions of mean pollutant-concentration distributions in a scaled model estuary characterized by steady-state salinity and velocity fields which are considered to vary only in the axial direction. Extension of the flushing theory to the prototype estuary is presumed to be valid.

Specifically, the theory may be used to predict the interval of time required to reduce a known concentration of pollutant to a stipulated level and to furnish, at integral multiples of tidal cycles, the approximate location within the estuary of the maximum pollutant concentration.

It is interesting that the assumed relationship between the eddy diffusivity and a characteristic length appears to be justified by the numerical results particularly as this would seem to imply that the relationship is of a universal character being valid for anisotropic turbulence as well as for isotropic turbulence. Since the concept basically involves energy and size, this result might be anticipated.

Eq. 37 takes the observed flushing anomaly,

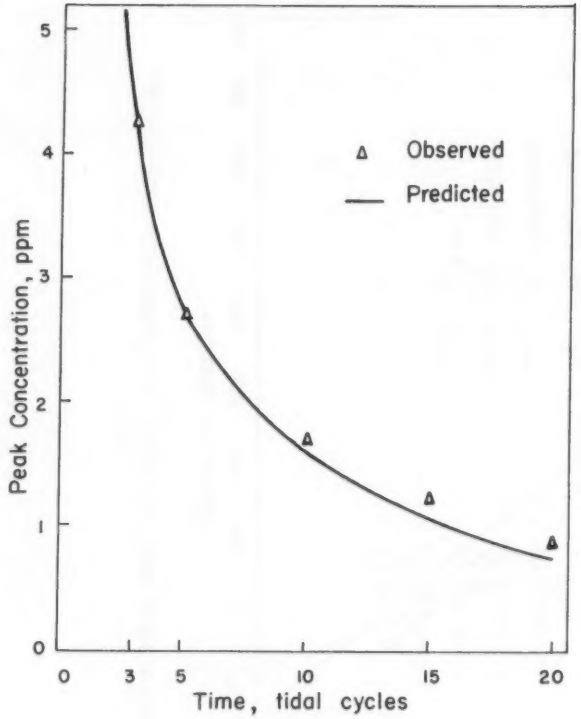

FIG. 9.—PREDICTED AND OBSERVED PEAK CONCENTRATION OF POLLUTANT VERSUS TIME FOR TEST I.

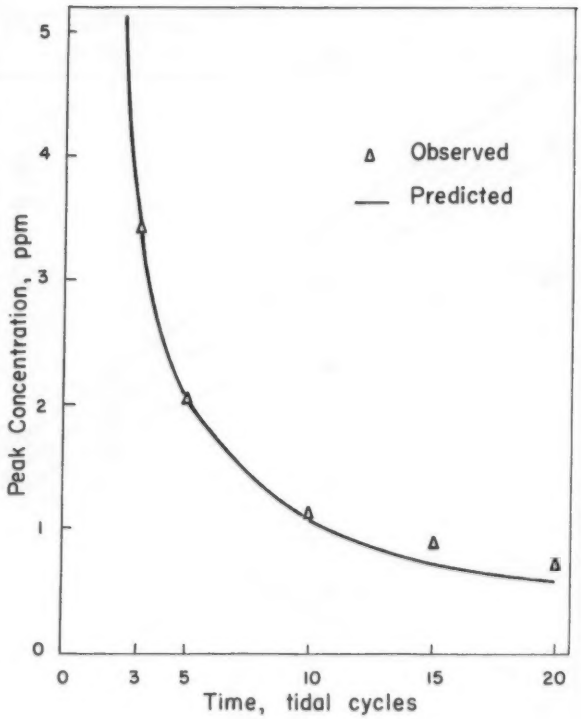

FIG. 10.—PREDICTED AND OBSERVED PEAK CONCENTRATION OF POLLUTANT VERSUS TIME FOR TEST II.

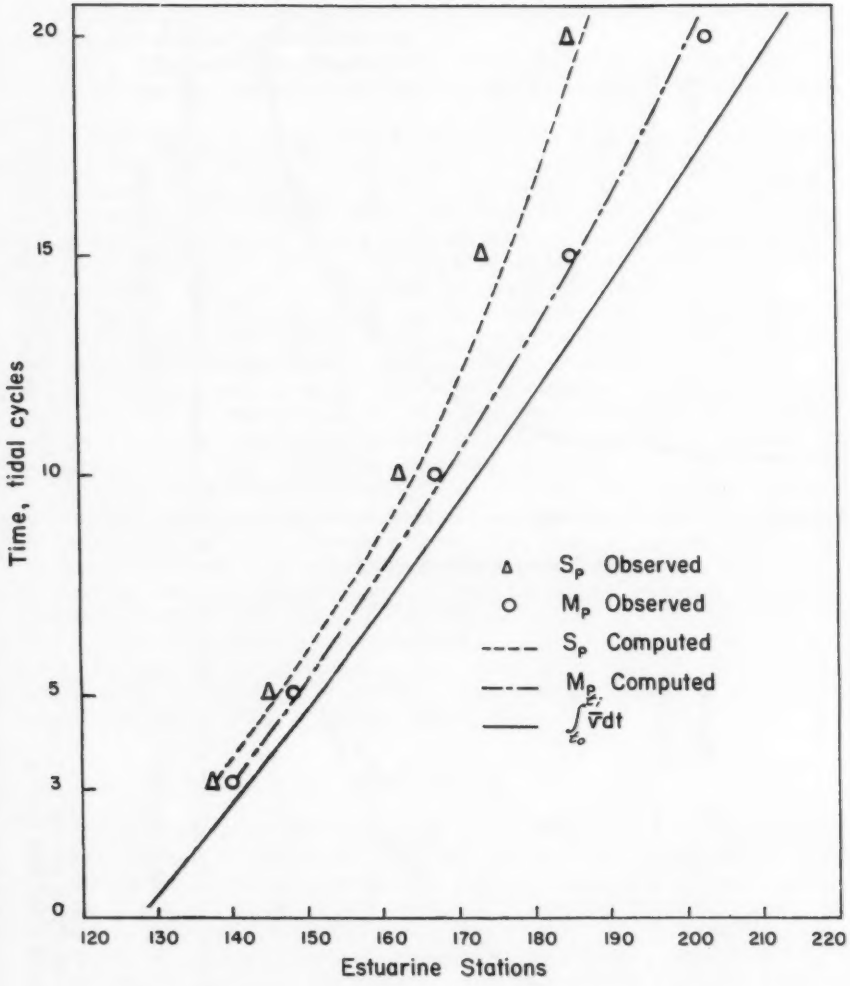

FIG. 11.—VALUES OF S_p, M_p, AND $\int_{t_o}^{t_i} \bar{v}\, dt$ VERSUS TIME FOR TEST I.

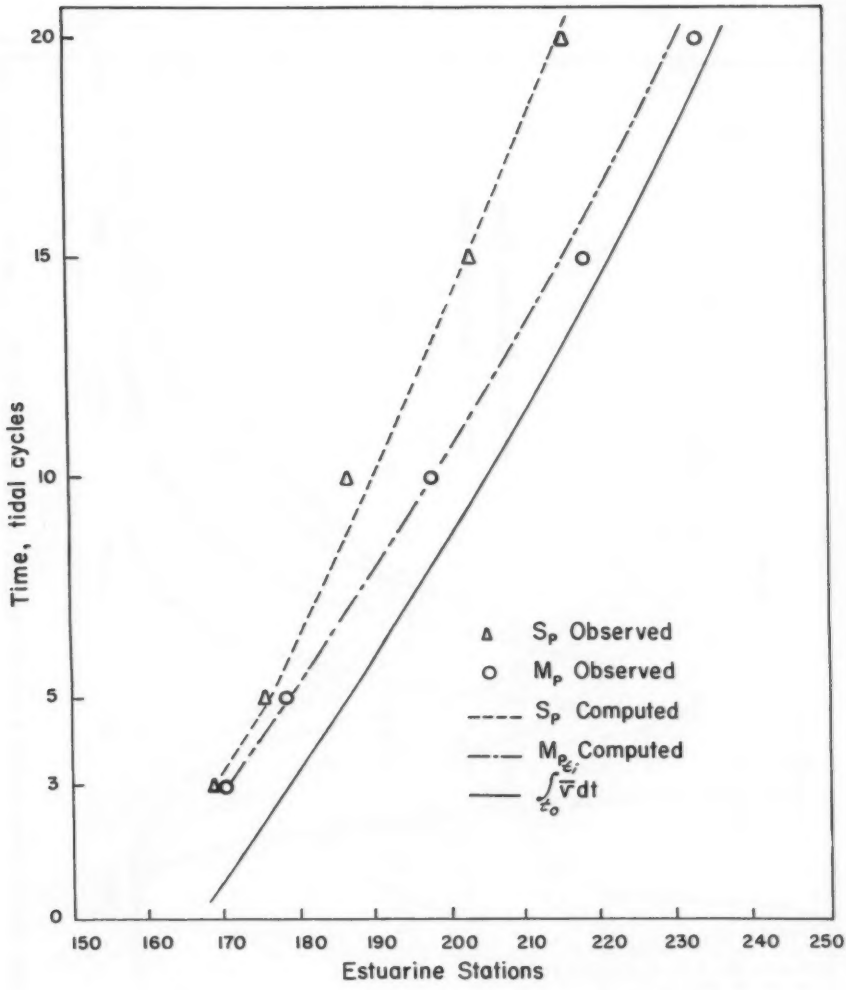

FIG. 12.--VALUES OF S_p, M_p, AND $\int_{t_o}^{t_i} \bar{v}\, dt$ VERSUS TIME FOR TEST II.

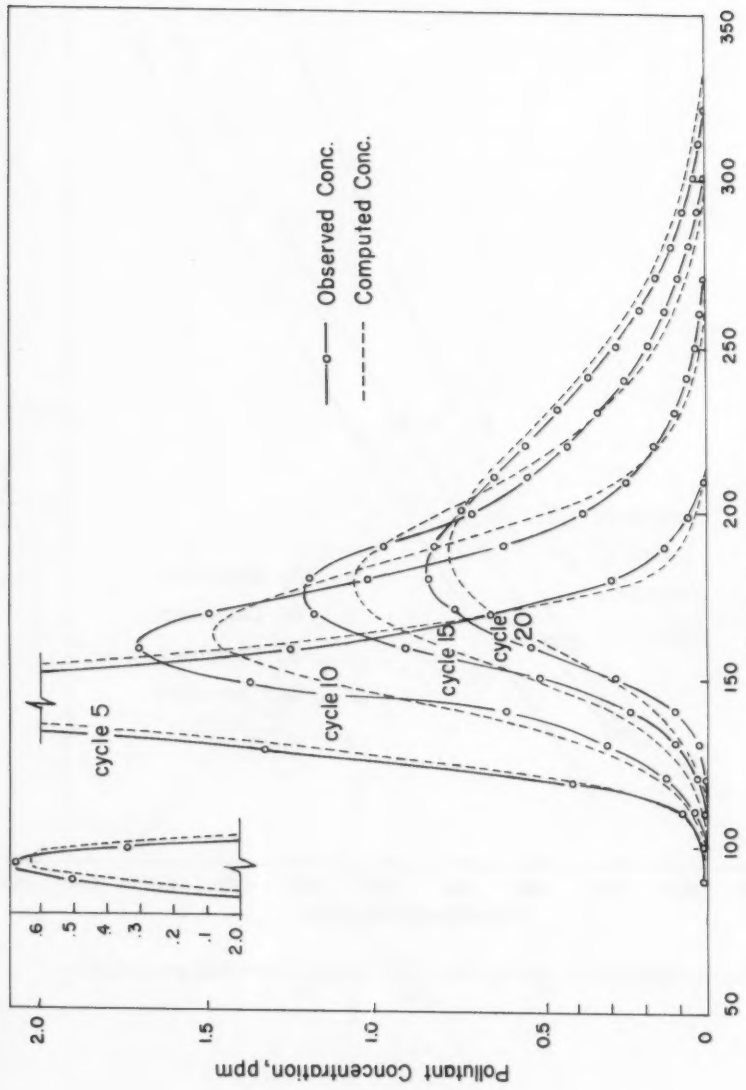

FIG. 13.—OBSERVED AND COMPUTED POLLUTANT CONCENTRATIONS FOR CYCLES 5, 10, 15, AND 20 OF TEST I.

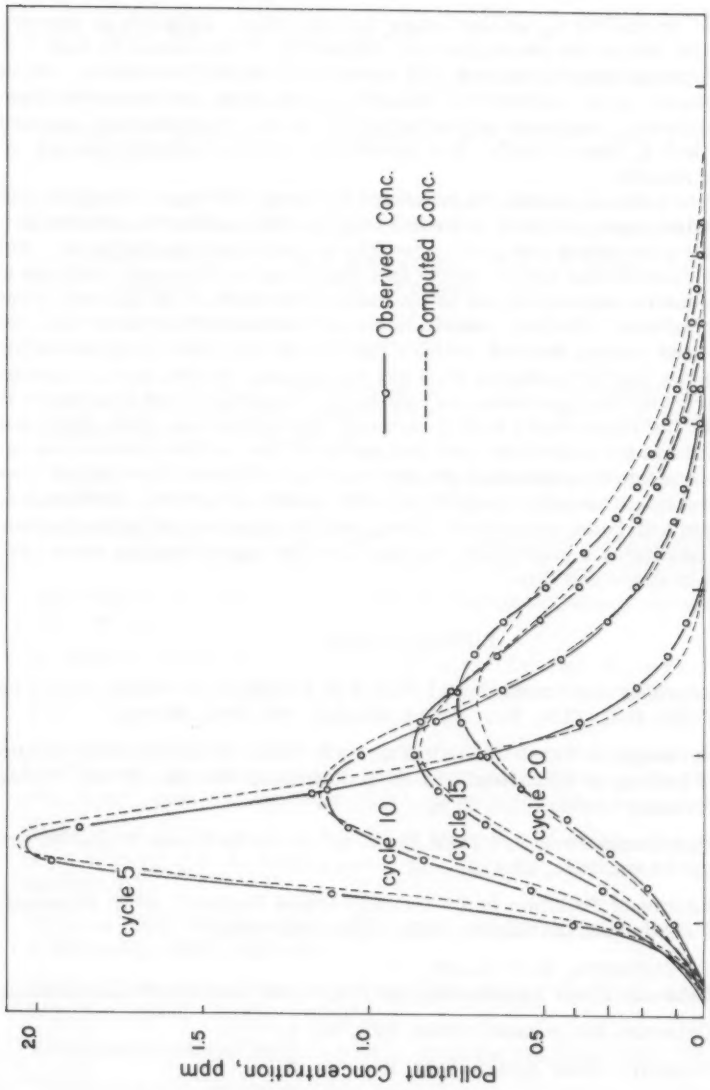

FIG. 14.—OBSERVED AND COMPUTED POLLUTANT CONCENTRATIONS FOR CYCLES 5, 10, 15, AND 20 OF TEST II.

$$\int_{t_0}^{t_i} \bar{v}\, dt - S_p > 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (66)$$

into account by making K_q an increasing function of x. Although no physical basis need lie behind the use of the eddy diffusivity, it is rational to expect an increase in the intensity of mixing with an increase in tidal velocities. Since in most estuaries it is common for the current amplitude and hence the free-stream turbulence to increase somewhat as the mouth is approached, the eddy diffusivity which, theoretically, is a measure of mixing intensity should increase accordingly.

Clearly, no general solution to problems involving turbulent transport, isotropic or anisotropic, is likely to evolve until the mechanics of turbulent motion is better understood and made amenable to mathematical synthesis. The situation is complicated further by the fact that those instruments available at present for micro-measurements in the natural regimen are of doubtful accuracy and precision. Indeed, measurements of turbulent flow made with the commonly used impact devices indicate that the Navier-Stokes equations for incompressible flow of a viscous fluid are not obeyed. Rather than to assume that the Navier-Stokes equations are inadequate, it seems more reasonable to suppose that the instruments tend to measure average values over space and time intervals which reflect the size and inertia of the devices themselves instead of the random or nonrandom properties of the turbulent fluctuations. Until theory and instrumentation capable of more closely describing distributions of property in a turbulent estuary are developed, the classical diffusion equation definitely constitutes a justifiable, practical tool for approximating mean concentrations in space and time.

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FUNDAMENTAL CONSIDERATIONS IN HIGH RATE DIGESTION[a]

By Clair N. Sawyer[1] and Jay S. Grumbling, A.M. ASCE[2]

SYNOPSIS

A comparison of the results obtained in high-rate digestion using shortened digestion times versus sludge concentration. The importance of fixed solids loadings in relation to mixing problems is described. Concepts of unimolecular and first order reaction kinetics are developed for anaerobic digestion.

High-rate digestion of sewage sludge has been studied extensively in laboratory, pilot, and plant scale operations by several investigators.[3,4,5,6,7,8,9,10,11] The senior author has summarized the status of knowledge as it existed in published form in 1957[12] and it is not the purpose of this paper to review such material but rather to summarize new information and concepts developed since that time.

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[1] Assoc. Engr., Metcalf and Eddy, Boston, Mass.
[2] Research Engr., Metcalf and Eddy, Boston, Mass.

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The investigations on high-rate digestion have established that there is one fundamental requirement, in addition to the normal ones, for the successful operation of high-rate units. This requirement is that the contents of the digestion tanks be thoroughly mixed on a more or less continuous basis.

Mixing accomplishes three major objectives all of which contribute toward keeping the biological forces operating at or near peak capacity at all times. First, the active organisms are kept continuously in contact with the food supply; second, the food supply is uniformly distributed and made as available to the organisms as possible; and third, the concentration of inhibitory biological intermediates and end products are maintained at minimum levels. All of these factors serve as a means of keeping the working population of organisms performing at peak efficiency.

From the engineering viewpoint it is not enough to just have the microbial population living in an ideal environment. Unless this advantage can be harnessed to do more than the normal amount of useful work, little has been gained. There is nothing the engineer can do, beyond maintaining an optimum environment that will change the rate at which individual organisms will work for him, other than to apply the well-established principles which govern first-order reactions. These are the principles that are applied, knowingly or unknowingly, when reduced detention times are employed to increase the organic loading on digestion units to as much as three or four times conventional. The same principles have been applied to the operation of high-rate trickling filters and the high-rate activated sludge process.

Sludge differs from sewage in several respects, but its one unique characteristic is the relative ease with which sludge can be concentrated under the influence of gravity. This property allows the engineer to increase organic loadings markedly without changing the detention time. Organic loadings can usually be increased by a factor of 1.5 to 2 by this practice alone, and usually result in improved operation of digestion units.

The ultimate in organic loadings is achieved by using a combination of the two systems mentioned. Fig. 1 illustrates the relative organic loadings that can be expected from each of the systems individually and in combination. The practicability of handling organic loadings in excess of 500 lb per 1,000 cu ft per day has not been demonstrated.

KINETICS OF SLUDGE DIGESTION

The degradation of organic matter in biological systems, aerobic or anaerobic, is brought about by micro-organisms, mainly bacteria, and the reaction may be represented for anaerobic systems as follows:

$$\text{Organic Matter} + \text{Bacteria} \rightarrow CH_4 + CO_2$$

Because two reactants are involved the reaction is binary in character but, since the bacteria are not consumed in the reaction, the reaction is actually first order in character, and the kinetics are the same as those for unimolecular reactions, as modified by the diverse nature of the organic matter and changes in bacterial populations. In a digester, which is being fed more or less uniformly on a day to day basis, it is reasonable to assume that the working population of bacteria is a constant. This leaves only the organic matter for consideration.

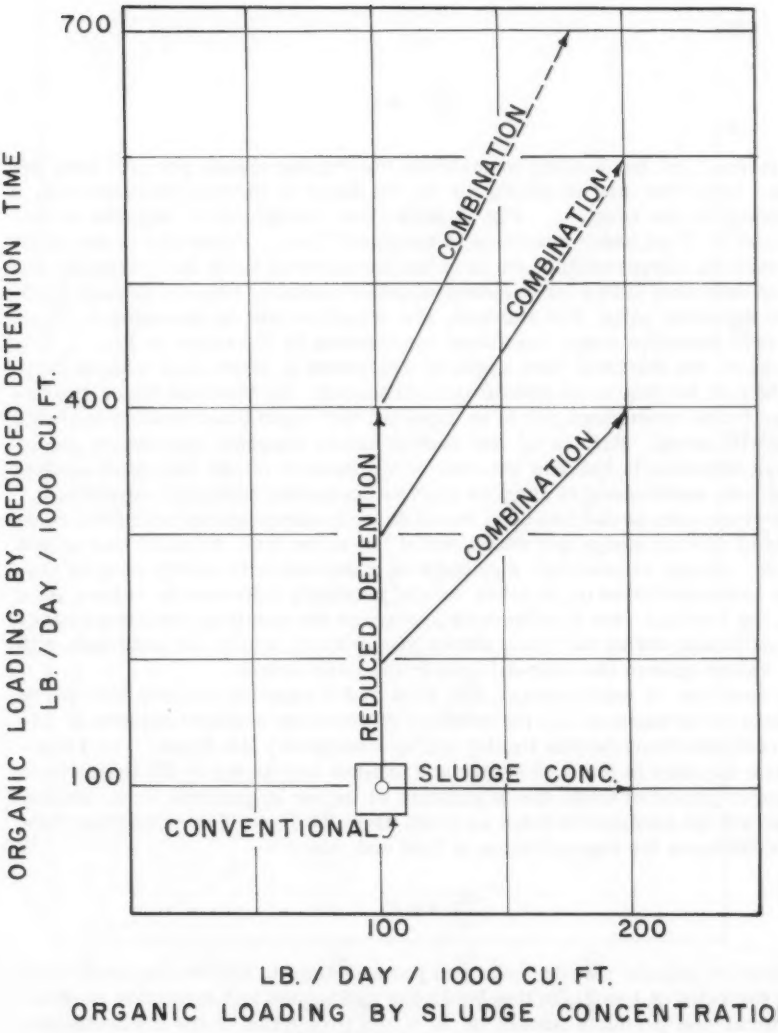

FIG. 1.—INFLUENCE OF SLUDGE CONCENTRATION AND/OR DETENTION TIME ON SOLIDS LOADINGS TO DIGESTION UNITS.

Studies of the kinetics of unimolecular and first order reactions have shown that the rate of such reactions is a function of the concentration of the reacting substance at any given time and may be expressed as follows:

$$\frac{-dC}{dt} \propto C$$

and

$$\frac{-dC}{dt} = k\,C$$

This means that the amount of material undergoing change per unit time decreases with time and is analogous to the decay of radioactive materials, a true unimolecular reaction. Fig. 2 shows the mathematical aspects of unimolecular or first order reactions in graphical form. From this curve or its mathematical interpretation, we have the fundamental basis for justifying decreased detention times as a means of accommodating greater organic loadings on digestion units. For example, if a digestion tank is operating at ten or more days detention under conditions represented by the curve in Fig. 2, it is obvious that the detention time could be decreased to seven days without serious effect on the degree of stabilization produced. As detention times are decreased below seven days it is to be expected that rapid deterioration in sludge quality will occur. Failure of the system can be expected long before one or two days detention is reached because of limitations of the biological system to cope with environmental changes such as decreased buffering capacity.

The discussion so far has been based on the assumption that all of the components of sewage sludge are stabilized at the same rate. Actually this is not the case. Sludge consists of a mixture of materials with widely varying biological assimilabilities or, in other words, markedly differend k- values. As a result, the reaction rate is influenced greatly by the easily assimilated high k-value materials during the early stages of digestion, while the materials with low k- values govern the over-all rate in the later stages.

The concepts of unimolecular and first order reaction kinetics also serve as a basis for predicting that the behavior of digestion units fed sludges of different concentrations (within limits) will be essentially the same. For example, if one digester is fed a 4% sludge and another unit is fed an 8% sludge over a prolonged period of time, the population of active organisms in the second digester will be essentially twice as great as in the first. The controlling factor then becomes the concentration of food and, since

$$\frac{-dC}{dt} = k\,C$$

the amount of organic matter destroyed per unit time in the two digesters will vary in the ratio of 1 to 2. On this basis gas production and formation of other biochemical end products should be in direct proportion to the concentration (%) of solids in the sludges fed, when all other variables are held constant. Another factor of some importance concerns the character of the digested sludge withdrawn from the units. For a given detention time, the digested sludge should be of essentially the same quality regardless of the concentration of solids in the sludge fed.

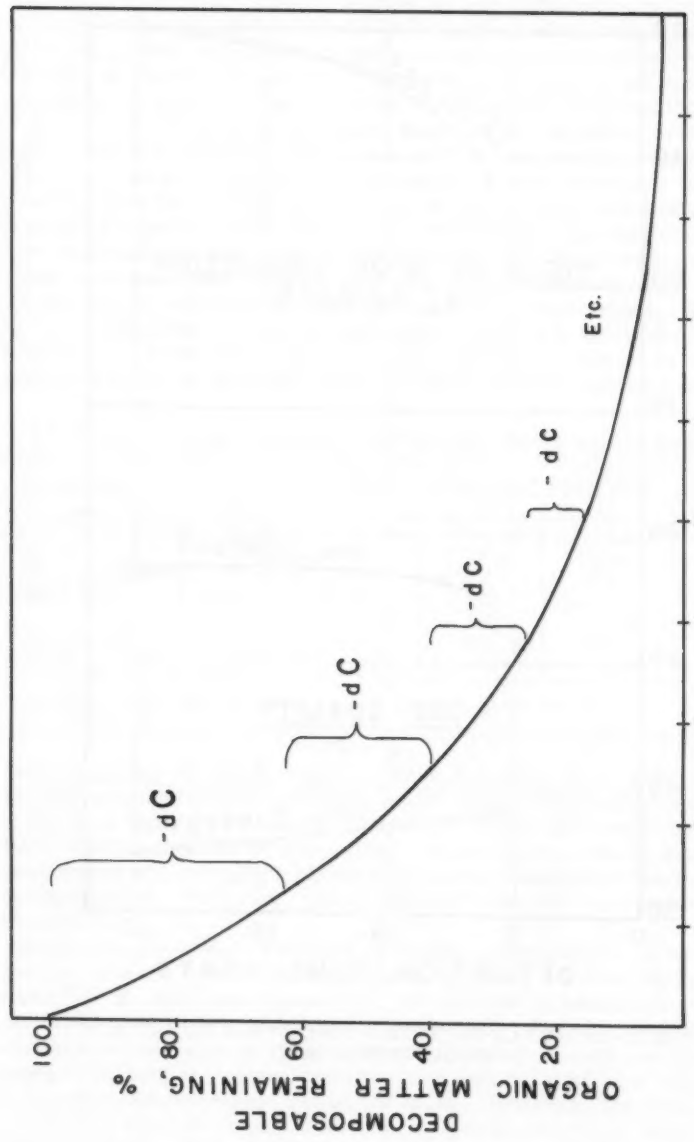

FIG. 2.—ILLUSTRATING THE KINETICS OF FIRST ORDER OF UNIMOLECULAR REACTIONS.

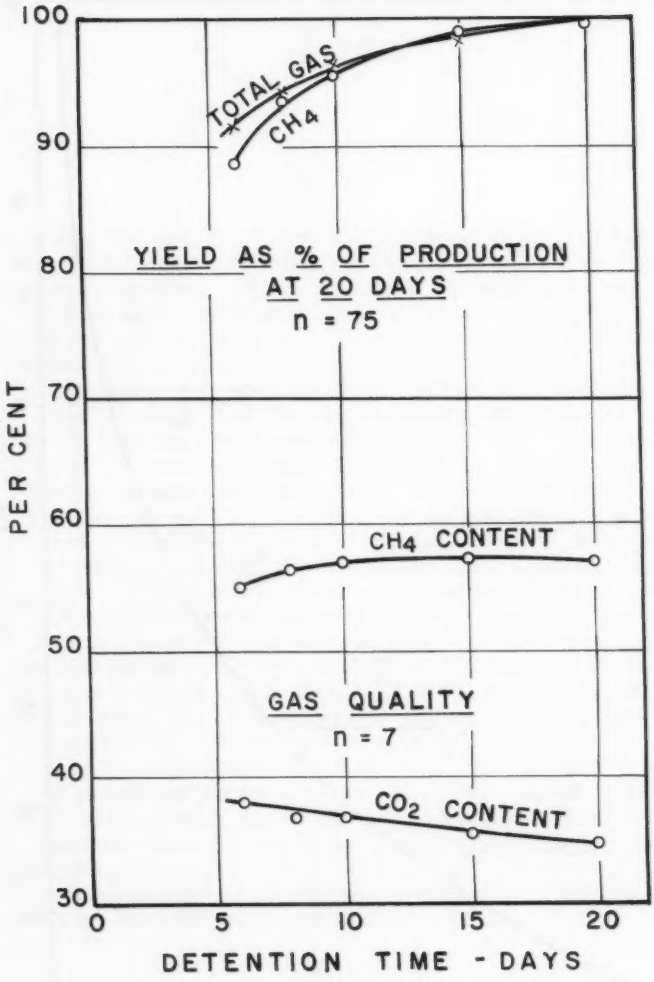

FIG. 3.—EFFECT OF DETENTION TIME ON GAS PRODUCTION AND COMPOSITION (REF. 7)

EXPERIMENTAL VERIFICATION OF THEORETICAL ASPECTS

Detention Time.—An extensive series of investigations were conducted by H. K. Roy[6] to determine the effects of detention times varying from 6 to 20 days on gas yields, composition of gas produced, volatile matter destruction, and quality of digested sludge. The results of his studies on gas yield and gas composition are shown in Fig. 3. From it, it may be noted that total production of gas and of methane decreased gradually as the detention time diminished. At six days detention the yield of methane and total gas were 88.8% and 91.6%, respectively, of the amount produced at 20 days detention. The trend of the curves from the tenth to the sixth day is increasingly abrupt and indicates that process failure could easily occur at detention periods less than six days.

The composition of the digested sludges withdrawn from the units is shown in Table 1. These data correlate well with the gas production values.

Of particular note are the volatile solids and grease values. When these values are converted to a percent destruction basis, the values given in Fig. 4 are obtained. From these data, we note that there is a marked decrease in grease destruction as detention times fall below 20 days, and the trend appears

TABLE 1.—COMPOSITION OF WITHDRAWN DIGESTED SLUDGE[a]

Determination	Digester Detention Period, Days				
	6	8	10	15	20
Total solids, %	3.15	3.05	2.99	2.92	2.77
Volatile solids, %	58.3	57.8	56.4	55.6	55.2
Volatile acids, ppm	270	140	110	110	110
Alkalinity, ppm	2,540	2,640	2,720	2,920	2,930
pH	7.0	7.05	7.15	7.30	7.45
Ammonia-N, ppm	400	418	434	464	465
Grease, %	9.50	8.85	8.05	7.15	6.65

[a] Based on 75-day period; average of 11 samples (from Ref. 6).

to be approaching a critical stage at a 6-day detention time. This is, undoubtedly, the major reason for the rapid decline in methane yield shown in Fig. 3. The trend of the volatile matter destruction curve appears to be much less critical but this is probably misleading. Actually, the volatile matter determination does not distinguish between biologically assimilable and nonassimilable substances. Therefore, interpretations from the curve should take this into account.

Sludge Concentration.—The effect of sludge concentrations on the anaerobic digestion process has been investigated on pilot or plant scale by W. N. Torpey,[9,10] F. ASCE and Brisbin.[13] The results of laboratory studies have been reported by H. Schmidt[7] and K. L. Schulze.[8] The data obtained by these investigators, particularly those of Schmidt,[7] have indicated that the degree of digestion obtained for a given detention time was more or less independent of the concentration of sludge fed to the units. However, the results of both Schmidt and Schulze suggest that there is a limitation to the concentration of sludge that can be fed and still maintain stable operation. Because of this possibility, and other reasons, investigations were started in our laboratory to study the subject in some detail.

[13] "Proceedings of Seminar on Waste Water Treatment and Disposal," by R. S. Rankin, Boston Soc. Civ. Engrs., 1957.

Laboratory equipment similar to that used by Roy,[6] utilizing gas recirculation for mixing, was employed for the studies. The digestion units have been fed once daily with sludges containing 4%, 6%, 8%, and 10% solids on a 15-day detention basis. Gas recirculation for mixing was intermittent (only 15 min of each hour). The sludge used was a mixture of primary and activated sludges obtained from the Leominster, Mass. sewage treatment plant. The sludge was concentrated by the system of flotation employed in the Laboon Process,[14] using a 24 hr or 48 hr contact period. Subnatant liquor from the flotation unit was used to dilute the thickening sludge to the proper concentrations for feeding the various units.

The data obtained on gas production and gas composition are shown in Fig. 5. These indicate that there was some deterioration in gas quality at the 10%

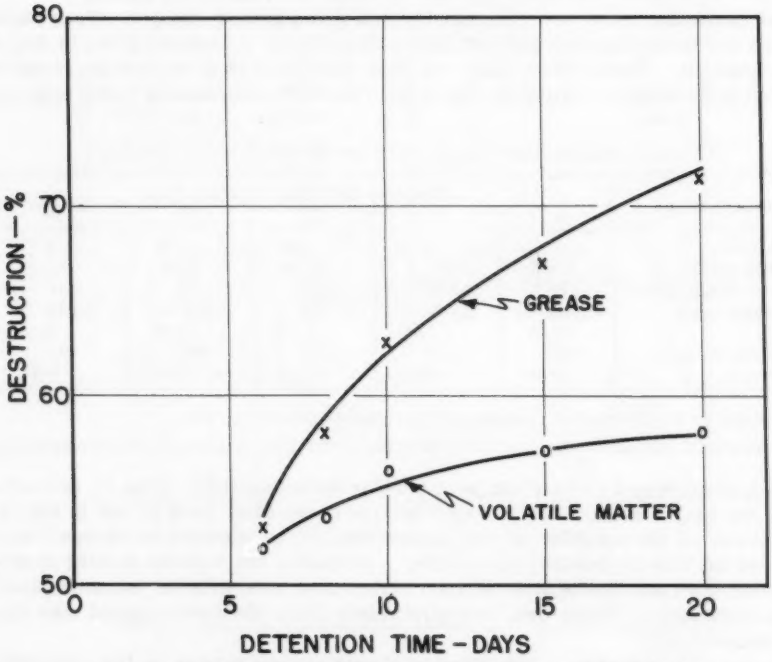

FIG. 4.—EFFECT OF DETENTION TIME ON DESTRUCTION OF ORGANIC MATTER (REF. 7)

solids level and that there was a significant decrease in total gas and methane yields as the concentration of solids increased in the sludge fed. The yield of methane for the 10% sludge was only 92% of that produced by the 4% sludge. This illustrates the point that concentration of sludges cannot be accomplished without some liquifaction of the sludge and loss of gas forming materials in subnatant or overflow liquors. It may be argued, however, that concentration by the Laboon Process gives an exaggerated condition because of the high temperatures and prolonged periods involved in the flotation units.

14 "Sewage and Ind. Wastes," by J. F. Laboon, 24, 423, 1952.

The data obtained on the digested material, as withdrawn from the digestion units, are shown in Table 2. The volatile solids and grease data show that the extent of destruction of organic matter was essentially the same in all units. The volatile acids and pH data indicate that very favorable conditions existed

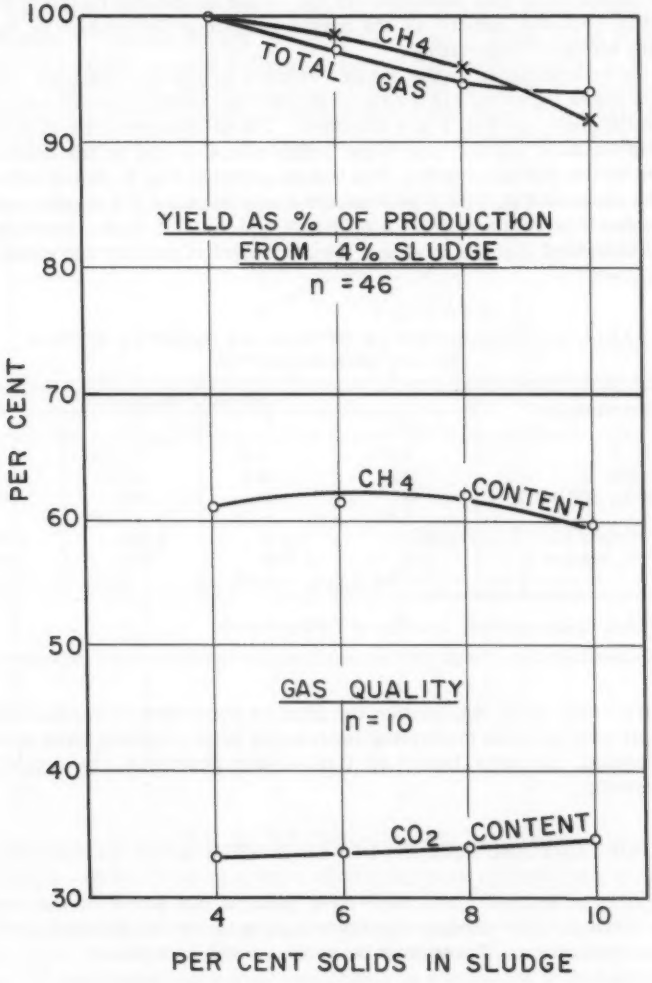

FIG. 5.—EFFECT OF SOLIDS CONCENTRATION IN SLUDGE FED ON GAS PRODUCTION AND COMPOSITION

in all units for proper functioning of the methane forming bacteria. The decreased methane content of the gas, therefore, must be attributed to other reasons. This is most likely due to the accumulation of nongaseous biochemical end products that remain in the digesting mixture. Some of these are indicated

by alkalinity and ammonia nitrogen in Table 2. It may be noted that the amounts of these materials increased as the concentration of solids in the sludge fed increased, but not in direct proportion. The reason for this anomaly is not evident at this time. Whether the accumulation of compounds containing ammonia or other biochemical end products are the cause of reduced methane formation and eventual process failure is the major subject of the research, now under way, in the writers' laboratory.

When the data obtained on the studies related to effects of sludge concentration (Table 2) are expressed in terms of percentage destruction of organic matter, the data shown in Fig. 6 are obtained. These show that the destruction of grease and volatile matter are quite independent of the solids content of the sludge fed to the digestion units. The values given in Fig. 6 should not be compared with those in Fig. 4 on a percentage basis because the sludge used in the latest studies was obtained from a different source. It was a mixture of primary and activated sludges having a lower content of grease and volatile matter.

TABLE 2.—COMPOSITION OF WITHDRAWN DIGESTED SLUDGE
(15-Day Detention Period)

Determination	% Solids in Sludge Fed[a]			
	4	6	8	10
Total solids, %	2.85	4.25	5.60	6.85
Volatile solids, %	54.0	55.5	54.5	54.5
Volatile acids, mg per l	40	50	80	105
pH	7.0	7.05	7.15	7.20
Alkalinity, mg per l	2,310	2,975	3,620	4,300
Ammonia - N, mg per l	555	740	910	1,090
Grease, %	6.10	5.67	5.65	5.86

[a] Based on a 52-day period; average of 7-13 samples.

Summary.—The data obtained on the studies conducted at various detention periods and with sludges containing increasing solids content have shown that the fundamental concepts based on first-order reactions are applicable to sludge digestion.

IMPORTANT FACTORS INVOLVED IN HIGH-RATE DIGESTION

The reports of Schmidt[7] and Roy[6] have pointed out some of the variables that occur in high-rate sludge digestion that may be considered as factors limiting its application. These may be summarized as follows:

1. Biochemical end products
   a. Increased alkalinity
   b. Increased ammonia
2. Decreased grease destruction
3. Decreased volatile matter destruction.

In the writer's studies, we have encountered difficulty in maintaining good mixing in the digestion units when the digesting mixtures contain over 6% total

solids. This is a factor which mitigates against the use of highly concentrated feed sludges, particularly sludges with low volatile matter content, and must be recognized if high-rate digestion units are to be properly designed and successfully operated. This factor brings into focus the importance of the fixed solids loading on digestion units as a design factor.

It has been recognized for some time that the digestibility of the volatile or organic matter in sewage sludge is a function of the volatile-fixed solids ratio. This matter was first brought to the attention of the profession by R. S. Rankin, F. ASCE in 1948[15] and is of such importance that his data are reproduced in Fig. 7. Experimental data obtained by Schmidt,[7] Roy,[6] and those in the current studies, have indicated that the volatile matter destruction during high-rate

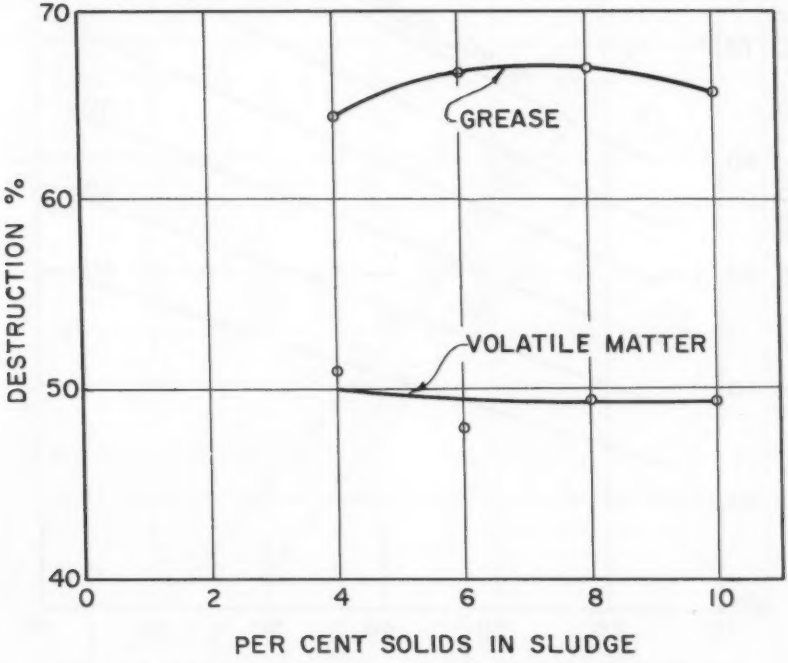

FIG. 6.—EFFECT OF SLUDGE CONCENTRATION ON DESTRUCTION OF ORGANIC MATTER.

operation, using a 15-day detention period, is equivalent to the destruction obtained in about 35 days under conventional conditions, and the destruction obtained in 10 days under high-rate conditions is equivalent to that obtained in 30 days under conventional operation. An interpretation of these data in terms of high-rate digester operation is presented in Fig. 8.

From the information shown in Fig. 8, it is possible to estimate the solids content of mixed liquors in high-rate digestion units, when fed sludges with varying percentages of fixed solids. This has been done for a unit operating on a 15-day detention period and the calculated relationships are shown in Fig. 9.

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15 "Sewage Works J.," by R. S. Rankin, 20, 478, 1948.

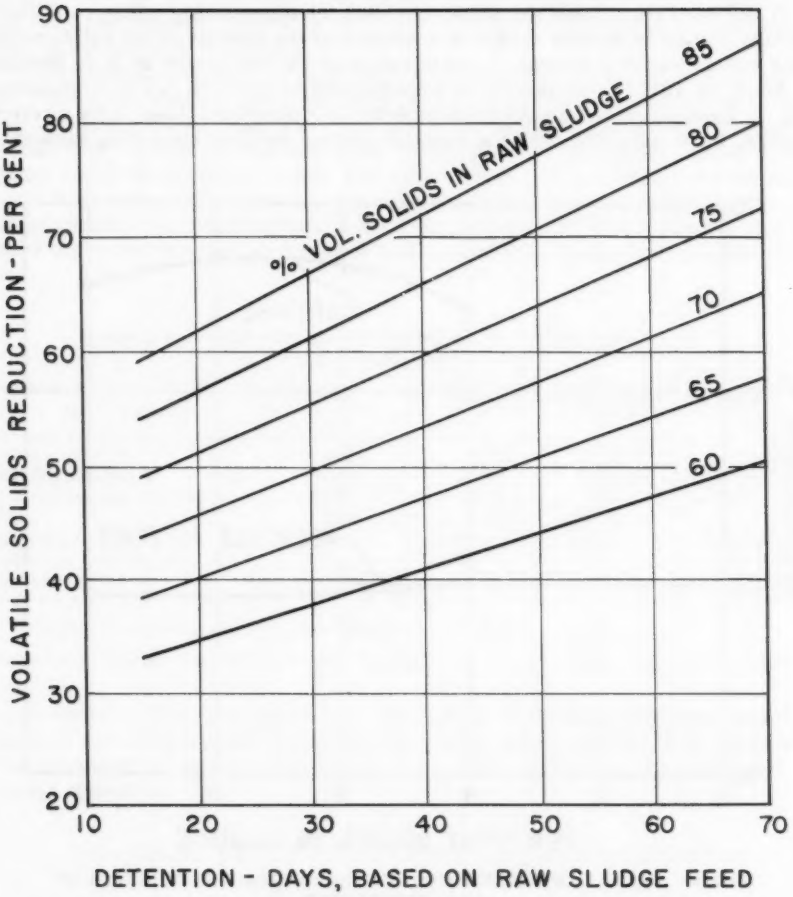

FIG. 7.—REDUCTION IN VOLATILE SOLIDS IN RAW SLUDGE FOR DETENTIONS FROM 15 TO 70 DAYS, T = 85° - 95° (REF. 5)

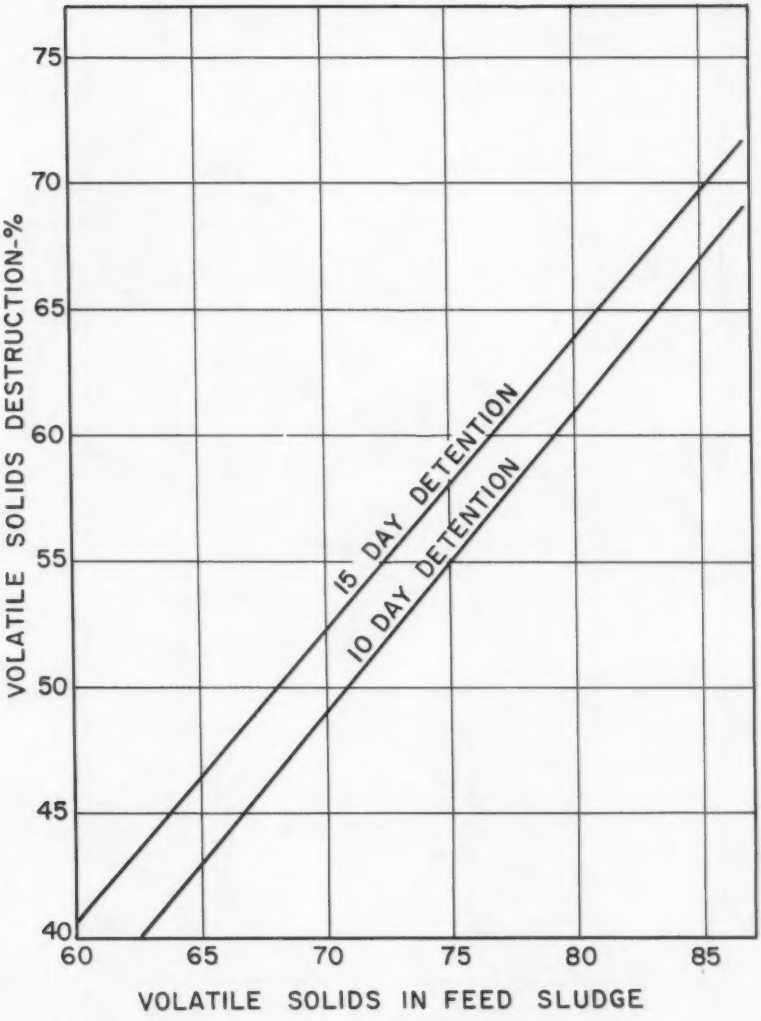

FIG. 8.—EXPECTED VOLATILE SOLIDS DESTRUCTION DURING HIGH RATE DIGESTION

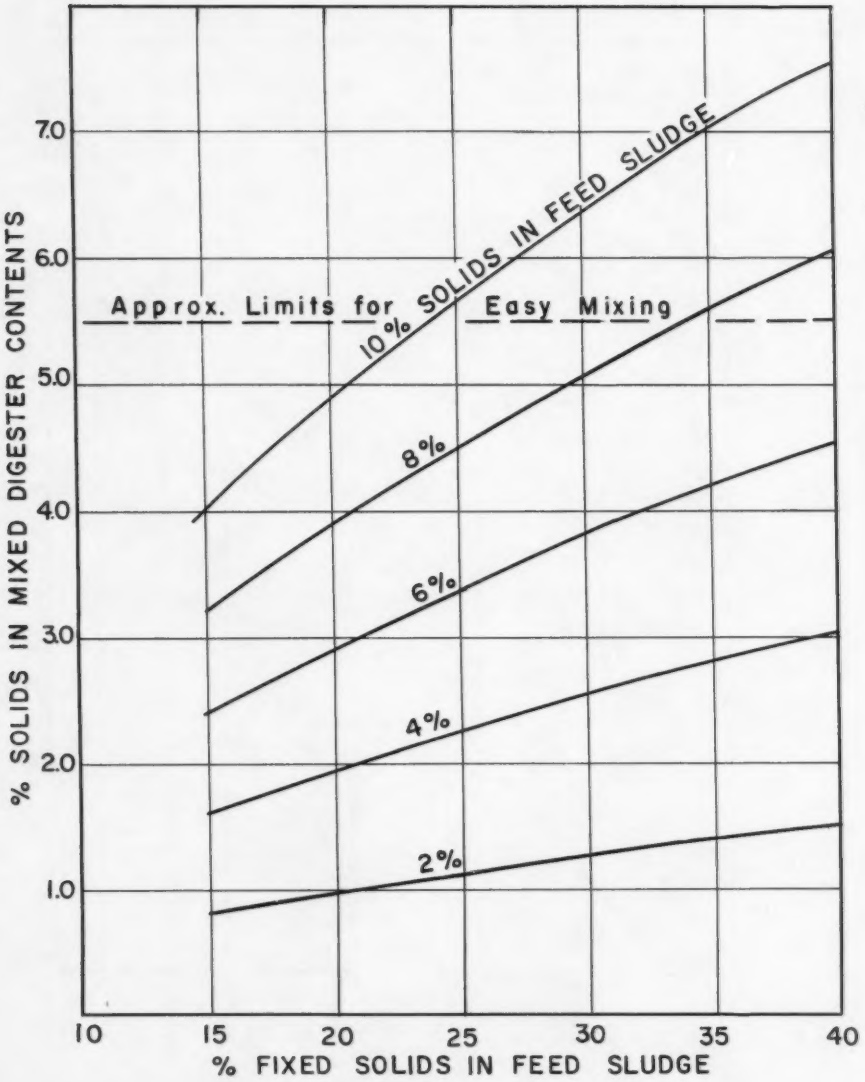

FIG. 9.—RELATIONSHIP BETWEEN SOLIDS CONTENT OF HIGH-RATE DIGESTERS TO FIXED SOLIDS IN FEED SLUDGE (15 DAY DETENTION)

The major requirement in high-rate digestion is adequate mixing. As the percentage solids in the digesting mixture increases, the viscosity increases or, in other words, the fluidity decreases to such an extent that it is very difficult to maintain proper mixing. Experience indicates that considerable difficulty is encountered when the solids content reaches 6%. This difficulty points up one of the major limitations of concentrating sludges; from the information given in Fig. 9, the writers conclude that it is impractical to thicken sludges to more than 8% solids unless the fixed solids content of the sludge is less than 35% (volatile > 65%). To add highly thickened low volatile content sludges to high-rate digesters will result in process failure, unless provisions are made for increasing the power and capacity of the mixing facilities.

CONCLUSIONS

1. The concepts of unimolecular and first-order reaction kinetics can be applied to high-rate digestion to predict effects of increased organic loadings by sludge concentration and/or reduced detention times.
2. Mixing of high-rate digesters becomes a problem as the total solids content of the mixed liquor approaches 6% solids.
3. A major parameter in the design and operation of high-rate digestion units is related to the fixed solids content of the raw sludge.
4. The total solids content of the mixed liquor in high-rate digestion units is a function of the fixed solids in the raw sludge and the detention time in the digester.
5. Feeding of highly concentrated, high fixed matter content sludges to high-rate digesters can cause process failure because of difficulties resulting from improper mixing.
6. Because of problems related to mixing, it appears impractical to concentrate feed sludges to more than 8% solids, unless the fixed solids content of the raw sludge is less than 35%, or the volatile matter content exceeds 65%.

ACKNOWLEDGMENT

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DIFFUSERS FOR DISPOSAL OF SEWAGE IN SEA WATER

By A M Rawn, Hon. M. ASCE,[1] F. R. Bowerman, A.M. ASCE,[2] and Norman H. Brooks, A.M. ASCE[3]

SYNOPSIS

For large sewerage systems the effectiveness of sewage disposal by dilution in the ocean depends on efficient dispersal of sewage into the receiving waters. By the use of large multiport diffusers on the ends of their outfalls, the Los Angeles County Sanitation Districts have, in recent years, greatly improved the characteristics of the ocean water near the disposal site despite steadily increasing discharges of primary effluent.

This paper includes a history of the ocean outfall system operated at Whites Point, Calif., by the Los Angeles County Sanitation Districts; an outline of certain techniques developed to predict dilutions of sewage effluent discharged in sea water; a description of the use of multiport diffusers for improving initial dilution; and a procedure for successful hydraulic design of large diffusers.

INTRODUCTION

The disposal of sewage effluent to bodies of water for dilution and final disposal requires an effective mingling of the effluent with the water body to

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[1] Chf. Engr. and Genl. Mgr., Los Angeles County Sanitation Districts, Los Angeles, Calif.

[2] Div. Engr., Los Angeles County Sanitation Districts, Los Angeles, Calif.

[3] Assoc. Prof. of Civ. Engrg., Calif. Inst. of Tech., Pasadena, Calif.; Consultant to Los Angeles County Sanitation Districts, Los Angeles, Calif.

achieve the following purposes: (1) the effective oxidation of the fine suspensoids and dissolved organic compounds; (2) the reduction of bacterial population; (3) the prevention of odor nuisance; and (4) the removal of particles by sedimentation with natural detrital bottom materials.

The disposal of sewage effluents into water bodies involves hydrodynamic relationships relating to the dispersal of one fluid into another; increased complexity results in the ocean from the substantial differences in fluid densities which cause the effluent to rise toward the surface of the salt-water body. For reasons to be cited subsequently, there are significant advantages in dispersing sewage effluent near the floor of the ocean as a means of increasing initial dilution. Some of the concepts embodied herein are applicable to the disposal of sewage effluent to freshwater bodies; however, disposal to the ocean is the primary subject of this presentation.

Where the favorable exchange of water through tidal or current movements exists, primary sewage treatment will usually afford adequate protection to the receiving waters, often without the necessity to chlorinate. Since primary treatment involves sedimentation processes which remove floatable constituents of sewage as scum from the surface of the sedimentation tanks while at the same time removing readily settleable solids from the bottom of the tanks, it can be seen that the primary treatment process eliminates the common complaints voiced against sewage disposal by dilution, to wit, the building of bottom sludge beds or the fouling of surface waters with floatable debris. No such similar protection is afforded by fine screening plants since these do not separate material according to their specific gravity, but only as to size. Floating rafts of fine sewage debris will be the almost inevitable result of the disposal of finely screened effluent into a body of ocean water.

The security of recreational ocean waters with respect to public health may usually be guaranteed through disinfection by artificial means such as chlorination, or by designing the outfall so as to prevent sewage effluents from reaching bathing areas in dangerous concentration. Where sufficient oxygenated diluting water is available to achieve initial dilutions of, say, 50 or more, there appears to be no need for more reduction of organic content of the sewage effluent than that provided by primary treatment, in order to prevent odors or other septic nuisances. Such dilutions are often found available where primary sewage effluent is proposed for discharge to ocean bodies.

OUTFALL SYSTEM OF LOS ANGELES COUNTY SANITATION DISTRICTS

The performance of a particular outfall or system of outfalls must be evaluated in terms of the physical circumstances extant at the site of the installation. No direct comparisons can, or should, be drawn between ocean outfalls unless consideration is given to the similarities and differences in the systems so compared.

Site Characteristics.—The Los Angeles County Sanitation Districts' outfalls extend off Whites Point, a small promontory on a peninsula of land extending into the Pacific Ocean southwestward from Los Angeles. The outfall site was chosen subsequent to work done by District engineers in the early 1920's; however, 6 miles of tunnelling under a range of low mountains was required to reach the shoreline, so that the first ocean outfall was not commenced until 1935 and not completed until 1937. At present (1960), the outfall system is disposing of the sewage from an area of almost 600 sq miles with a

population of more than 2,500,000. Characteristics of the outfall site are as follows:

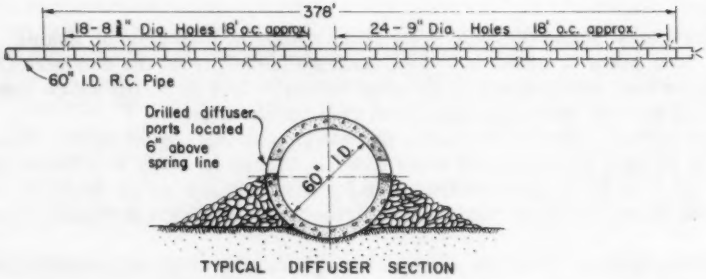
1. The rocky Palos Verdes Peninsula, comprised largely of shales and schists, juts about 5 miles into the Catalina Channel from the Southern California coastline, terminating at its most offshore face in a cliff which rises a distance of 80 ft or more abruptly from the ocean's edge.
2. The bottom, offshore from the peninsula, extends seaward at a slope of about 1 in 50 as a wave-washed ledge of rock, becoming sandy at a water depth of about 30 ft or 40 ft, with detrital muds commencing at a water depth of about 80 ft to 90 ft, remaining as mud bottom further offshore into the Catalina Channel.
3. Strong littoral currents sweep the promontory, running predominantly parallel to the shoreline, with occasional weak currents of short duration moving toward shore.
4. Weak winds, usually from the west, cause movement of surface water toward shore, but have as their principal effect the establishment of a mass transport of water such as to induce upwelling of cold bottom water near the shore. The favorable effect of this latter phenomenon will be investigated subsequently.

The Outfall System.—At Whites Point, the Los Angeles County Sanitation Districts have constructed three ocean outfalls; one built in 1937, 60 in. in diameter and extending 5,000 ft from shore to a depth of 112 ft; a second, built parallel to the first, in 1947, having an internal diameter of 72 in, originally terminating the same distance from shore, but later lengthened to 6,800 ft from shore in 1953, at which time a diffusion structure was also constructed on the line; and a third, 90 in. in diameter, completed in 1956, extending 8,000 ft from shore to a depth of 212 ft, terminating in an even more efficient diffusion structure than that placed on the 72-in. outfall. The hydraulic capacity of the three pipes is about 500 mgd. The outfall system is designed for disposal of primary effluent, depending on natural dispersion and oxidation in the receiving waters. The outfalls are shown in Fig. 1 and 2.

The three ocean outfalls have been built as trenched conduits through the rock section of the ocean's floor, becoming semiflexible, articulated pipelines laid on the surface of the sand and mud at greater ocean depths.

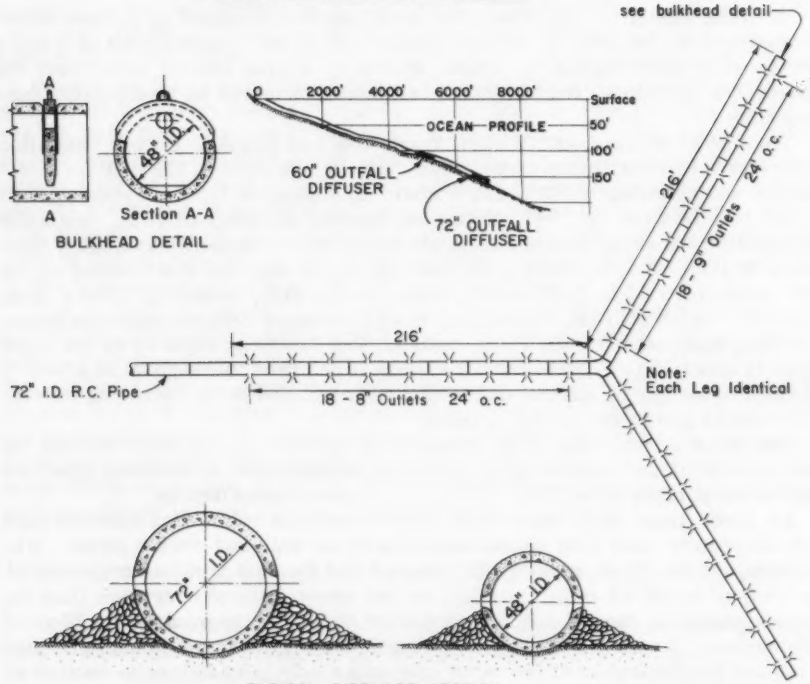
All three lines have been constructed from cast reinforced concrete pipe with meehanite cast iron tongue-and-groove or ball-and-socket joints. Examination of the 60-in. pipe in 1957 showed that the cast iron had corroded at the rate of about 10 mils annually, so that penetration of corrosion (that is, graphitization) of about 0.2 in. has occurred during the approximately 20 yr of submersion. No significant loss in strength has yet been occasioned, although continued graphitization at the same rate may cause joint failure in another 40 yr or 50 yr. The most recently placed outfall has sacrificial magnesium anodes attached to the cast iron end rings. Concrete cores bored from the 60-in. diameter outfall show that the crushing strength of the concrete exceeds that measured more than 20 yr ago.

Effluent Disposal Without Diffusers.—In 1933, the Districts commenced a routine observation of the ocean environment and measurement of bacterial populations. An intensified survey program was commenced in 1949, where, in addition to visual observations and bacterial analyses, the following factors were measured:


378'
18 - 8" Dia. Holes 18' o.c. approx.
24 - 9" Dia. Holes 18' o.c. approx.
60" I.D. R.C. Pipe
Drilled diffuser ports located 6" above spring line
60" I.D.

TYPICAL DIFFUSER SECTION

DIFFUSER FOR 60" OUTFALL



TYPICAL DIFFUSER SECTION

DIFFUSER FOR 72" OUTFALL

FIG. 1.—OCEAN OUTFALL DIFFUSERS

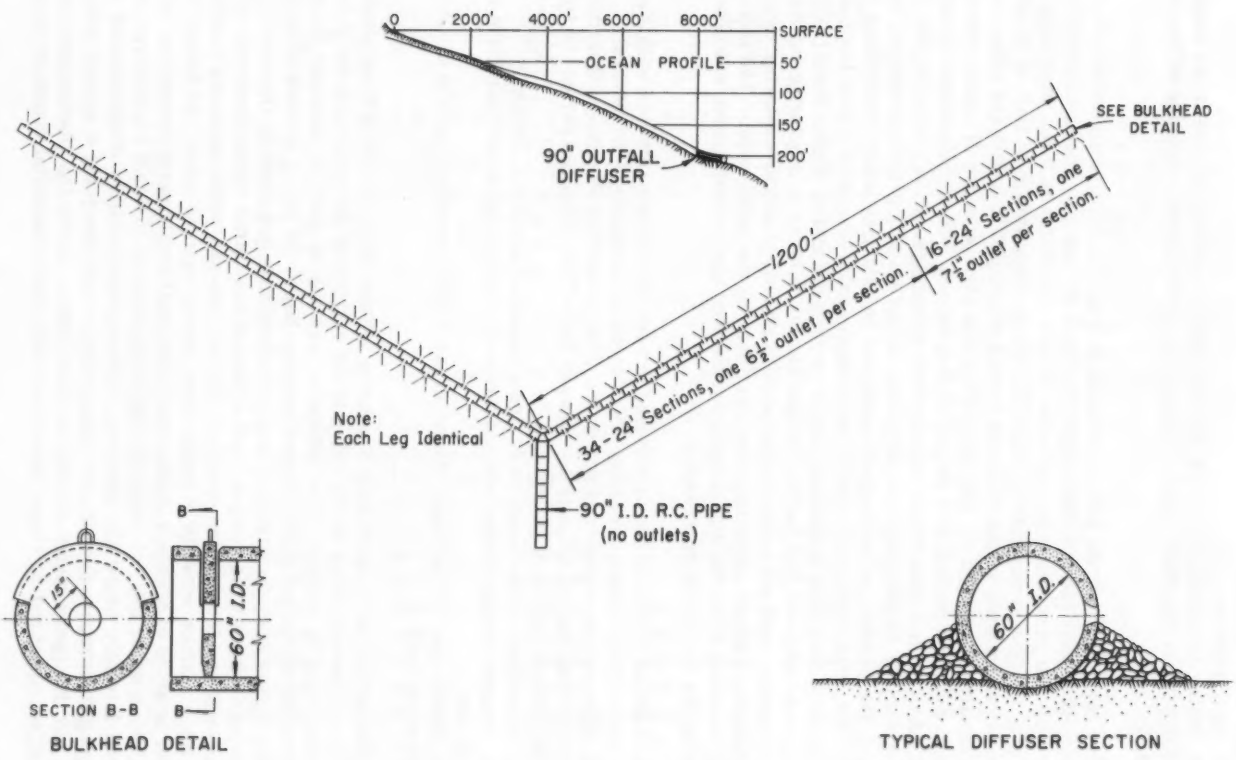


FIG. 2.—DIFFUSER FOR 90-IN. OUTFALL

1. Ocean currents in the vicinity of the point of discharge at the surface and at a depth of 10 ft;
2. Wind velocity; and
3. Water temperature at the surface and at various depths at two points; one 5,000 ft and the other 7500 ft offshore along the line of extension of the outfalls.

The level of bacterial activity is seen in Fig. 3 to be directly related to the amount of sewage effluent discharged; in Fig. 3 the bacterial concentrations are based on the percentage of bacterial counts (coliform M.P.N.) exceeding 10 per ml. California regulations establish for bathing waters a limit of 10 coliform organisms per ml as the level beyond which public health and safety may be impaired, qualified by the stipulation that the frequency of counts greater than 10 coliform per ml may reach, but not exceed, 20% of the number of samples taken, based upon twenty consecutive samplings. It is presumed that the occurrence of coliform counts exceeding 10 per ml at a greater frequency than 20% of the number of readings is sufficient to establish some credence in the data. By 1952, bacterial levels were approaching the point where improvements were indicated in the disposal of sewage effluent at Whites Point (Fig. 3) and it was indicated that further thought should be given to improving the mixing of sewage with sea water at the point of discharge and to exploring design modifications which might improve conditions at the outfall site. The sewage field resulting from the discharge from the then open-ended pipes was objectionable for the following reasons:

a. Surface layers of poorly diffused sewage sometimes traveled with shallow, wind-driven ocean currents, which move predominately shoreward;
b. Mixtures of sewage and sea water, often exceeding 2% sewage, were quite stable and maintained identity as "sewage fields" for extended periods of time and for several miles of travel;
c. Shoreline coliform counts at times exceeded allowable limits;
d. Visible sewage fields created areas of discolored water seen from the shore; and
e. Poorly diffused sewage effluent created odor problems, both in and near the floating field.

Effect of Ocean Temperature Gradients.—Experience at Whites Point showed coliform concentrations at the shore in the vicinity of the outfalls to be higher in winter than in summer. The difference was due, in part, to stronger shoreward winds occurring during winter months, but in the main, seemed to be influenced greatly by the stronger ocean-temperature gradients in the summer.

Sewage effluent discharged from a deep ocean outfall mixes first with cold bottom water. It was often observed that sewage-sea water mixtures reached the ocean surface some 10°F colder than nearby surface water. At times, the colder mixture of sewage and sea water was sufficiently more dense than surface ocean water that, on reaching the ocean surface, it would resubmerge and spread laterally below the surface, a phenomenon readily demonstrated by dumping fluorescein dye into the rising column and observing it spread laterally and disappear under nearby surface water. Since the phenomenon just described appeared at times coincident with lower bacterial counts at shore

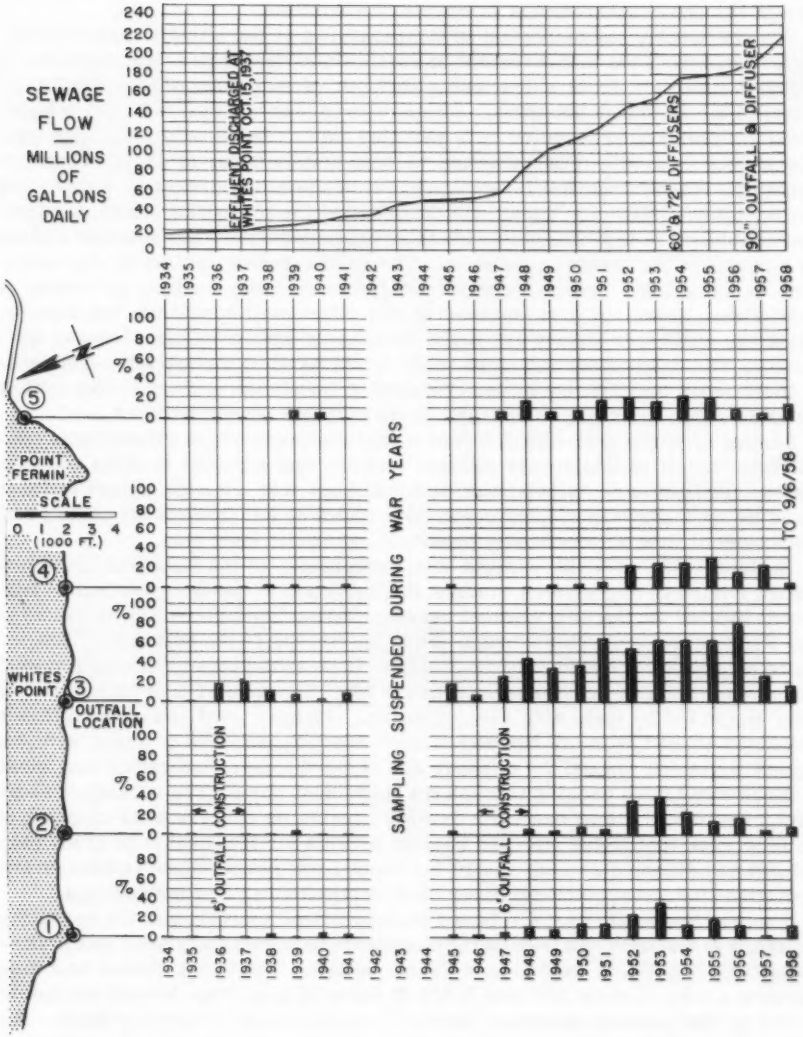


FIG. 3.—BACTERIAL CONCENTRATIONS AND RATE OF SEWAGE EFFLUENT DISCHARGE

sampling stations, both effects being observed during the summer and fall, it was deemed advisable to measure more carefully some of the physical properties of the ocean environment in search of a solution.

To the weekly measurement of temperatures at two specific locations offshore was added the measurement of surface and subsurface temperatures at 1,000-ft intervals from shore along the line of extension of the outfalls to a distance of 10,000 ft seaward. Temperatures were measured with a highly sensitive thermistor mounted on a sounding cable and the data plotted on profile sheets (Fig. 4). The presence of cold bottom water at shallower depths during the summer and fall is accounted for by upwelling of bottom water along the Southern California coast, resulting from the prevailing westerly winds. Warm surface waters are quite naturally explained during the summer and fall as a result of the greater insolation. The winter-spring regime is characterized by very little change in water temperature from surface to bottom in near-shore water, in part because of the absence of upwelling, but also because the shallower waters are more completely mixed by the stronger wave action. The deep-water diffusers, built by the Sanitation Districts at the ends of their ocean outfalls, have been designed to invade the area of colder bottom water in order to create submerged fields of dilute sewage effluent.

Studies of Diffusers.—Small-scale model studies confirmed theoretical conclusions that a multiple-port diffuser system was superior to other systems tested. Preliminary calculations demonstrated that a multiple-port diffuser could lie on a seaward sloping ocean floor and flow full through all ports at all conditions of flow without costly penalty in hydraulic head loss.

In July 1953, with the consent and cooperation of the East Bay Municipal Utility District, the writers studied the operation of the Oakland Outfall Diffuser, located in the ship channel between Oakland and Yerba Buena Island in San Francisco Bay. The average depth to the top of the horizontal, single-barreled, multiport diffuser is about 40 ft. Digested sludge is discharged with the sewage effluent for dispersion in the bay. Because of high turbidity, the diver inspected by feel, with zero visibility. Inshore sections of the diffuser evidenced about 1/4 in. of "greasy slime" around the inside of ports, with the deposit lessening toward the offshore end of the diffuser, indicating that most of the flow had been passing through the innermost ports. The buildup of sediment, grease, and slime was very limited, causing no concern over clogging of the pipe, even though the pipe had been in service for the discharge of primary effluent and digested sewage sludge for about 1 yr. Recent observations on the Sanitation Districts' outfall diffuser have verified these earlier findings.

Construction of Diffusers.—Based on theoretical analysis and the operating structure at Oakland, diffusion structures were designed and placed on the Districts' existing 60-in. outfall and 72-in. outfall. The 72-in. outfall was discharging a flow of about 100 mgd 5,000 ft seaward and it was deemed advisable to convey that quantity of sewage farther from shore and to greater depth. An additional 1,500 ft of 72-in. diameter pipe was added to the outfall, extending the structure to 6,500 ft seaward in 150 ft of depth. Fig. 1 shows the design of the 60-in. diffuser and the 72-in. diffuser.

A single-barreled diffuser was deemed adequate for the 60-in. outfall, discharging 50 mgd at 1,100-ft depth. The original three-outlet cast-iron diffuser placed at the seaward end of the 60-in. outfall was closed with steel plates, leaving only a small opening at the top and bottom of the central port to provide some self cleansing. Commencing with the first inshore pipe section from the cast iron diffuser, 9-in. diameter holes were bored slightly above the spring

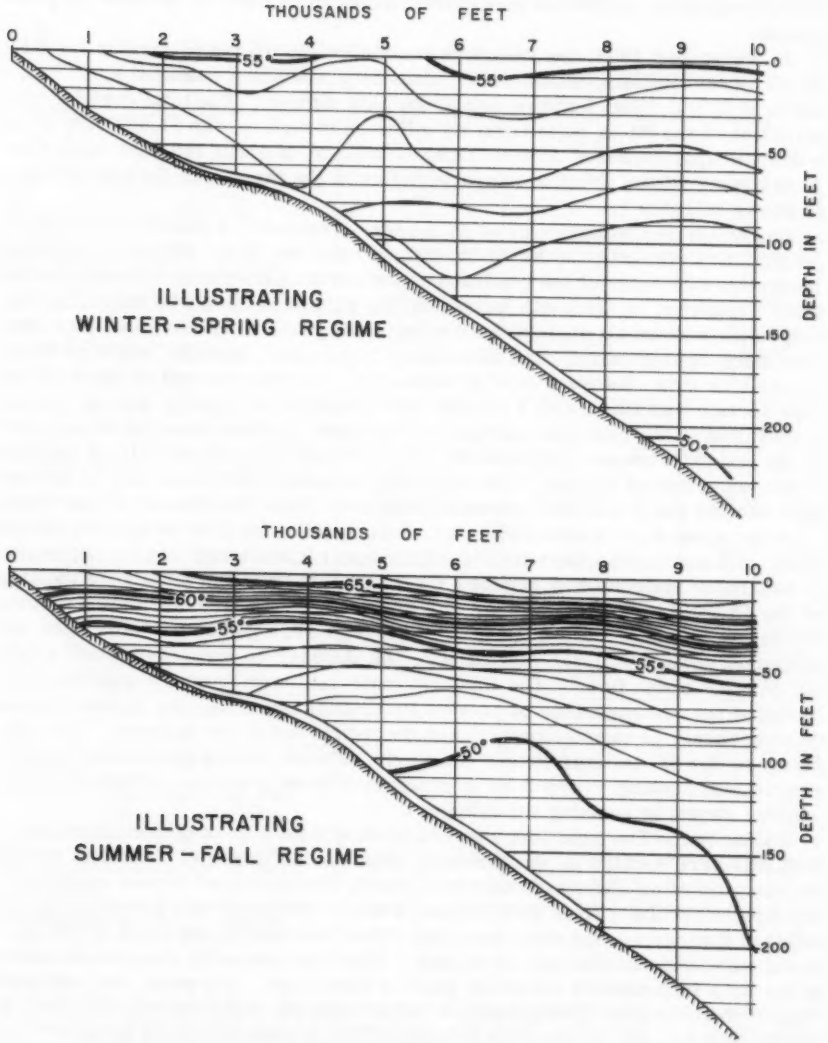


FIG. 4.—OCEAN-TEMPERATURE PROFILES

line on opposite sides of the pipe, two in each 18-ft pipe section (Fig. 1). The additional head loss for the diffusers did not exceed 2 ft at peak flow. The operational experience gained from the two diffusers during 1954 and 1955 provided confirming data for the multiport diffuser to be used on the next proposed outfall.

In November 1956, the Districts completed and placed in service the third of its ocean outfalls, 90-in. inner diameter, extending seaward 8,000 ft to a depth of 212 ft; Fig. 2 shows details for this diffuser. The total cost for construction of the 90-in. outfall and its diffusion structure was $1,686,000; it has a design capacity, based on dispersing efficiency, of about 150 mgd mean flow. A summary of the physical characteristics of the three outfalls and diffusers is shown in Table 1.

Inspection and Maintenance of Diffusers.—Following a year of discharge of the primary effluent thru the 60-in. diffuser and the 72-in. diffuser, a physical inspection was made of the condition of the ports to determine if sedimentation were occurring in the main barrel of the pipe or if the ports were clogging. Inspection showed all ports to be flowing; side ports in the terminal pipe section were approximately half filled with "soap-like" deposits which partially blocked the flow, but were readily dislodged. The bottom half of the terminal pipe section was filled with a deposit which tapered so rapidly that it was not discernible in the next pipe section. There was no flow from the bottom slot in the end gate; however, sewage effluent was found to be flowing from the hole in the upper end of the gate. The observed deposits were found only in the last pipe section and it was not deemed necessary to clean the diffuser at that time.

After about 4 yr of operation, a careful inspection (by free-swimming divers using self-contained underwater breathing apparatus) showed that the deposition of settleable material had reached the third port from the termini of the legs of the 72-in. line and the diffuser was cleaned. Using free-swimming divers, the bulkheads which closed the ends of the legs were jacked partially open and sufficient sewage effluent passed thru the diffuser to eject the settled solids out onto the ocean floor. The floatable material blown from the pipe was collected at the ocean surface to prevent its drifting onshore. No further action was necessary to satisfactorily scour the debris out of the diffuser. The diffuser on the 90-in. outfall was similarly cleaned without incident; a routine program of flushing every 2 or 3 yr appears to be a simple, relatively inexpensive means of cleaning the diffusers.

Evaluation.—The experience gained from almost 5 yr of operation of manifold diffusers located at considerable depth in an ocean environment having regular and rapid change of water has clearly demonstrated certain maeliorating effects resulting from such diffusers when compared with previous poorly diffused discharge. One very desirable effect has been a reduction in the bacterial population in near-shore waters. The most dramatic changes occurred in the area immediately above the point of discharge. Whereas, well-defined "boils" were readily distinguishable before adequate diffusion was initiated, it has become difficult to detect the location of the sewage discharge by other than chemical or bacteriological means, except on infrequent occasions during the winter when discolored sea water appears over the diffusers during periods of high flow. Oceanographic studies made prior to the use of diffusers could readily depend on the sewage bubbles as points of reference; with the diffusers installed, it has become necessary to establish off-shore locations by use of sextant, as only in this manner can one be assured of sampling from an area directly over the point of discharge, or at some predetermined distance from the outlet.

In addition to the disappearance of the sewage bubbles, the sewage field or "sleek" has become so indistinct as to be indiscernible to other than a practiced eye. Density differences are reduced to the point that wave patterns

TABLE 1.—SUMMARY OF DIFFUSER CHARACTERISTICS

	Outfall		
	1	2	3
Outfall inside diameter, in inches	60	72	90
Year outfall placed in service	1937	1947	1956
Year diffuser placed in service	1954[a]	1953	1956
Length of ocean outfall excluding diffuser, in feet	4,500	6,400	7,900
Total length of diffuser, in feet	384	648	2,400
Average depth of discharge (below mean sea level), in feet	108	155	203
Number and diameter of ports	26 at 9 in.[b] 18 at $8\frac{3}{8}$ in.	40 at 9 in.[b] 18 at 8 in.	2 at 15 in. 32 at $7\frac{1}{2}$ in. 68 at $6\frac{1}{2}$ in.
Shape of ports	circular (sharp-edged)	circular (bell-mouthed)	circular (bell-mouthed)
Ratio: $\frac{\text{Total port area}}{\text{Outfall cross section}}$	0.94	0.85	0.63
Spacing of ports, in feet	2 every 18 ft	2 every 24 ft	24
Design peak flow, mgd	90	150	260
Calculated additional head loss due to diffuser[c] at design peak flow, in feet	1.9	2.1	6.1
Design friction factor in diffuser, f	0.026	0.030 (legs) 0.026 (stem)	0.024
Manning roughness n, equivalent to f-values above	0.0155	0.0155	0.015

[a] Diffuser constructed by bulkheading end of outfall and drilling holes in main pipe.
[b] Includes 2 slots in each end bulkhead, each slot being equivalent to 9 in. diameter port. [c] Includes pipe friction in diffuser but not density head due to changes in depth.

move through the dilute sewage-sea water without discontinuity; reflective and refractive properties are so little changed as to make it almost impossible to see the sewage field.

Prior to the installation of diffusers, the area immediately over the points of discharge smelled faintly, but distinctly, of sewage and industrial wastes and, at times, the odor was wind-borne for as much as 1 mile. With diffusion,

the problem of odors has ceased except during calms when a faint odor may be detected directly over the discharges. With even the slightest breeze this odor is so quickly dissipated as to be indiscernible a distance of 1,000 ft. The vicinity of the sewer outlets has, for many years, been a favorite fishing grounds for a number of commercial sports-fishing boats that depend on the attraction of the sewage effluent for an assurance of a fair day's catch of bottom fish. Since the use of diffusers, these fishing craft have been noted to station themselves at times directly over the diffusers.

Benthic studies in the vicinity of Whites Point disclose that ocean plant life has ceased for a distance of about 1/2 mile in all directions from the termini of the diffusers, possibly caused by toxic effects of industrial wastes, lessened salinity, or a decrease in transmitted sunlight (necessary for plant growth) resulting from increased turbidity in the immediate vicinity of the sewage discharge. Since the operation of the diffusers, the surface turbidity immediately over the diffusers has decreased; on occasion, the turbidity has actually been less than that of the surrounding ocean waters. The explanation for this apparent anomaly seems to lie in the fact that much of the diluting water is drawn from the bottom layers of ocean water; turbidity from sewage effluent, added to the very clear bottom waters, apparently results in a less turbid mixture than that of the surface waters which usually contain plankton.

The effect on the concentration of coliforms in shore waters is not so easily demonstrated as is the effect on ocean water in the immediate vicinity of the point of discharge. Of advantage is the fact that submerged sewage fields generated by the diffusers are subjected to shoreward currents which are weaker than those at the surface, allowing more time for natural reduction of coliform concentrations. Additionally, much higher initial dilutions have been achieved with diffusers, although this advantage is partially lost because the dilution rate occurring in dilute sewage fields formed by diffusers is slower than that occurring in the narrow field originating from an open pipe. For these reasons the benefits to be derived from diffusion will depend on a great many factors other than the simple relationship between the sewage concentrations in diffused or undiffused fields.

The general pattern of improvement in density of coliforms has met expectations (Fig. 3). It is not possible to make direct comparisons between numerical results since many factors obscure the pattern of coliform distribution resulting from the discharge of sewage effluent through the diffusers: to wit, the continuing increase in discharged sewage effluent, overlapping effects of discharges from local systems, and the disturbance of the ocean floor with depth charges in the construction of outfalls.

Averaged data on the concentration of sewage effluent in the ocean immediately overlying the points of discharge have been compared in Table 2 for the periods prior to and subsequent to the use of each diffuser. The diffuser on the 90-in. diameter outfall has produced a sewage field averaging about 1 part sewage effluent to 400 parts of sea water during the winter-spring regime when the sewage field is at the surface; when the field is submerged surface samples taken directly over the diffuser yield negative results, both for chloride changes and for the presence of coliform bacteria; no measurements have as yet been taken of the dilution obtained in the submerged fields.

Summary.—The outfall system of the Los Angeles County Sanitation Districts has provided a case history spanning more than 20 yr of experience in the disposal of a primary sewage effluent to a favorable ocean environment. Since 1953, the installation and operation of deep-water multiport diffusers

has provided comparative experience indicating major reductions in odor, discoloration, and turbidity, all being significant improvements in the aesthetic acceptability of the discharge, as well as measurable reductions in bacterial populations. Inspection and maintenance of the diffusers have been accomplished in depths to 212 ft without difficulty.

DILUTION IN A RISING JET OF SEWAGE

To analyze horizontal discharge from an ocean outfall, assume that sewage effluent is discharged near the ocean floor from the end of a single pipe or port. Most important are determination of the rate of dilution of the sewage as it rises toward the surface of the salt water and the shape, density, and other characteristics of the jet or "rising column" of sewage. Discharge from a vertical or an inclined port or outlet will not be discussed herein because under such circumstances the effluent is given vertical momentum initially, thus

TABLE 2.—IMPROVEMENT IN INITIAL DILUTION BY USE OF DIFFUSERS AT WHITES POINT

Outfall	Dilution[a]	
	Without Diffuser	With Diffuser
60-inch	40	170
72-inch[b]	30	300
90-inch	. . .[c]	400

[a] Based on average of maximum sewage concentrations at ocean surface over point of discharge, determined by ammonia concentration measurements. [b] 72-in. outfall was extended from depth of 110 ft to 155 ft at time diffuser was constructed. [c] Never operated without diffuser.

rising to the surface faster and in a shorter path without other compensating influences.

Except as indicated the ocean current is assumed to be nil, and ocean salinity and temperature are assumed uniform.

Description of the Problem.—When the sewage flows into sea water from the end of a pipe, it is immediately subjected to a buoyant force proportional to the difference in density between the sewage and the surrounding sea water. This force deflects the sewage jet, or stream, toward the surface and accelerates the sewage upward. The relative motion between the sewage jet and the sea water develops shear stresses. Turbulence is generated and mixing takes place first around the periphery of the jet or rising column and finally throughout the whole column. As the mixing progresses, density differences are decreased and the vertical force driving the sewage effluent toward the surface gradually decreases. Since the mixing is greatest around the periphery, that volume is decelerated most rapidly, and velocity gradients are developed across the column, causing shear and turbulent diffusion throughout the entire column. From energy considerations, it can be shown that the upward velocity of the column

as a whole gradually decreases as it rises, except possibly for a short distance at the start of its rise where differential densities are sufficient to accelerate the motion of the sewage effluent.

There is a complex interrelationship between the buoyant gravity force and the turbulent shear force. Both forces depend on the nature of the flow pattern, yet the flow pattern is determined by the forces acting; added to this dilemma are the uncertainties of the turbulent mixing process.

The appearance of the rising column is shown in Fig. 5 which is a photograph of a small jet in a laboratory tank. It is not surprising that there should be some difficulty in a direct physical analysis. The flow is inherently fluctuating due to its turbulent nature. Where a column of rising sewage effluent breaks the surface of the ocean, the so-called "bubble" shifts back and forth at random, making it impossible to identify any fixed point as the center; diluted sewage effluent arrives at the surface in numerous small "gusts." Divers report that large shifting eddies comprise the periphery of the column. At best, only time averages of the actual phenomenon can be considered.

The flow from an ocean outfall is essentially that of a submerged jet with buoyancy, with the flow field limited by the water surface and the ocean bottom. For a submerged jet without density difference in an infinite or semi-infinite fluid field, there are no external forces, such as gravity, affecting the flow; the only force is the internal shear. Consequently, considering the whole region of flow, the total momentum flux of the flow is constant. On the other hand, when external gravity forces deflect the jet into a curvilinear path, the momentum flux is no longer constant, and there is no simple way in which the buoyant force can be fitted into the equations for momentum flux. Likewise, the elementary principles of continuity and conservation of energy cannot be applied to the rising column. The volume of upward flow is constantly increasing due to the dilution, and energy is dissipated in turbulence.

A comprehensive experimental study of the hydrodynamics of submerged jets with buoyancy is needed. Some outstanding studies of diffusion of submerged jets without gravity effects have been published; especially noteworthy is the work of M. L. Albertson, Y. B. Dai, R. A. Jensen, and Hunter Rouse, whose analysis[4] is based on reasonable theoretical hypotheses and experimentally determined constants. It may be possible to extend their analysis to the more general case with a gravity force. Also of interest is the ingenious theoretical solution by G. Abraham[5] for the case of vertical discharge with buoyancy in an infinite medium.

Without the benefit of comprehensive research, one recourse is to resort to model experiments and empirical curve fitting. A M Rawn and H. K. Palmer[6] did this for both horizontal and vertical jets. The empirical formula derived from their experiments for dilution at the surface applies only to one particular set of units because the equation is dimensionally nonhomogeneous. With a somewhat different approach, utilizing dimensional analysis, it is possible to develop relationships in terms of dimensionless parameters. Further-

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4 "Diffusion of Submerged Jets," by M. L. Albertson, Y. B. Dai, R. A. Jensen, and Hunter Rouse, Trans. ASCE, 115 (1950), p. 639-697.

5 "Diffusion in Submerged Circular Vertical Turbulent Water Jets with Density Difference Between the Jet and the Surrounding Water," by G. Abraham, Tech. Rept. 138-2, Hydraulic Eng. Laboratory Wave Research Projects, Univ. of Calif., Berkeley, August, 1959.

6 "Predetermining the Extent of a Sewage Field in Sea Water," by A. M. Rawn and H. K. Palmer, Trans. ASCE, Vol. 94 (1930), pp. 1036-1081.

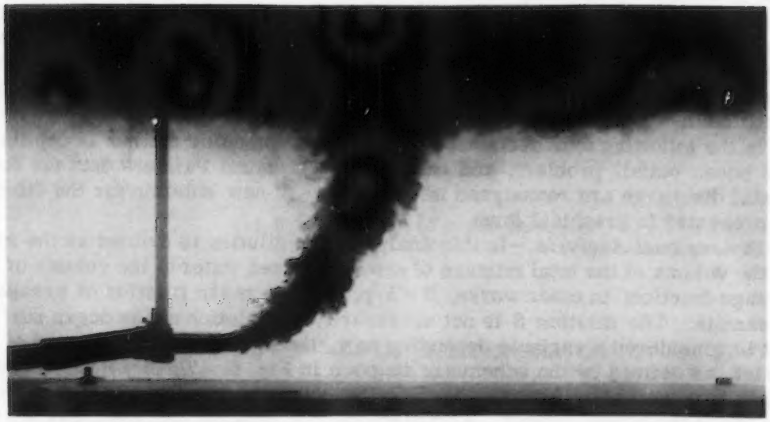

FIG. 5.—RISING COLUMN OF DYED FRESH WATER IN A SMALL LABORATORY TANK OF SALT WATER

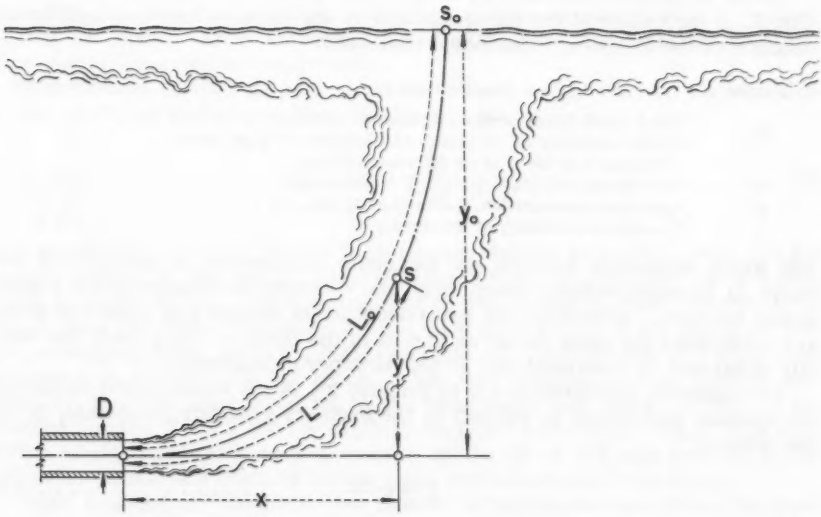

FIG. 6.—SKETCH OF SINGLE RISING SOLUMN SHOWING NOMENCLATURE AND COORDINATE SYSTEM

more, without considering some of the familiar dimensionless hydraulic numbers, questions arise as to the range of conditions over which the empirical formula is valid and as to the degree of similarity of flow patterns in model experiments to those of prototype cases. In addition, where dimensionless numbers are used, direct comparisons with other hydraulic phenomena are facilitated.

In the following four sections, the theory of hydraulic models is applied to the ocean outfall problem, and the original Rawn and Palmer[6] data for horizontal discharge are reanalyzed in this light. A new solution for the dilution is presented in graphical form.

Dimensional Analysis.—In this analysis, the dilution is defined as the ratio of the volume of the total mixture of sewage and sea water to the volume of the sewage fraction; in other works, $S = 1/p$, where p is the fraction of sewage in a sample. The dilution S is not necessarily the dilution at the ocean surface but is considered a variable depending on y, the height above the center of the outlet, as defined by the schematic diagram in Fig. 6. When $y = y_o$, the total depth from center of the outlet to the water surface, then $S = S_o$ the dilution at the top of the rising column. Furthermore, the dilution will also vary across the jet, becoming greater toward the periphery; hereinafter, S will denote to the minimum dilution, which occurs along the axis of the jet. An additional complication is that turbulent fluctuations will cause the dilution at any one point to vary considerably with time, but since these fluctuations are not of especial importance, the analysis will deal only with the time-average dilutions.

In the simplified case with no temperature gradients or currents, the dilution S_o in the center of the rising column at the surface may be considered a function of the following independent variables:

Symbol	Definition	Dimensions
y_o	Total depth from center of outlet to surface	L
D	Initial diameter of jet (same as diameter of pipe, port, or nozzle if there is no jet contraction)	L
V	Jet velocity $4Q/\pi D^2$, where Q = discharge	L/T
g'	Apparent acceleration due to gravity (Eq. 1)	L/T^2
ν	Kinematic viscosity of the sewage	L^2/T

The small difference between the kinematic viscosities of sewage and sea water is inconsequential; hence, only one viscosity is included in the dimensional analysis. Moreover, the mass densities of sewage and receiving water are considered the same as far as inertia of the fluids is concerned; the density difference is important only in the buoyancy calculations.

The apparent acceleration due to gravity, g', is the acceleration caused by the buoyant force, and is related to the ordinary gravitation constant, g, by the relation

$$g' = g \frac{\Delta s}{s} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which s is the specific gravity of sewage and Δs denotes the difference in specific gravities of sea water and sewage.

Provided the discharge pipe is raised slightly above the ocean floor at the outlet end, as is usual, the actual distance from the center of the outlet to the bottom appears to be an unimportant variable in most practical cases, because the jet quickly turns upward away from the ocean bottom.

Including $S \rightarrow S_o$, which is dimensionless, there are six pertinent variables, involving two fundamental units, length and time. Therefore, by the Pi Theorem, there are 6 - 2 = 4 independent dimensionless numbers which can be formed from these six variables. It is most convenient to choose them as follows:

(a) S_o

(b) y_o/D

(c) $F = \dfrac{V}{\sqrt{g' D}}$ (Froude number) ..... (2)

and

(d) $R = \dfrac{V D}{\nu}$ (Reynolds number) ..... (3)

Thus S_o may be considered a function of the other three parameters,

$$S_o = f(y_o/D, F, R) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

where F and R are the familiar Froude and Reynolds numbers in hydraulics.

The Reynolds number used is for the jet at its origin, and characterizes the relative importance of viscosity. For $R > 2{,}000$, the jet flow will usually be turbulent, but the turbulence is probably not fully developed until R reaches 10,000 or 20,000. As R increases still further, the kinematic nature of the flow may be expected to change little.

The Froude number gives the relative influence of gravity on the jet. For a free liquid jet discharging horizontally into air,

$$F = \frac{V}{\sqrt{g D}} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

For small values of F, the jet is deflected sharply by gravity, whereas for large values of F, the jet has more momentum and the trajectory turns downward more gradually. For a submerged jet the Froude number, as defined in Eq. 2, has the same significance as it does for the case of the liquid jet into air, assuming here, as in the discharge into air, that friction is a secondary influence in determining the shape of the flow pattern near the point of discharge. For the submerged jet in the laboratory tank shown in Fig. 5, the Froude number F = 2.2 and the depth to diameter ratio $y_o/D = 15.3$. This condition could correspond to a prototype discharge of 139 mgd from an 84-in. diameter outfall at a depth of 108 ft, using Froude-number similarity.

With dimensional analysis, the number of independent quantities has been reduced to three, each of which has a particular physical significance. However, the actual functional relationship between S and the three variables y_o/D, F, and R must be determined experimentally.

Hydraulic Similitude and the Use of an Hydraulic Model.—If the three parameters are identical for two distinct cases, then S_o must also be identical for the two cases. The fundamental requirement of any model study is to make the relevant dimensionless numbers the same in the model as in the prototype.

Geometric similarity is certainly necessary, but far from sufficient. However, it is virtually impossible in any model study to achieve similarity for both Froude and Reynolds numbers simultaneously;[7] this is found to be the case in the ocean outfall problem under consideration.

For a submerged jet with no buoyancy, Albertson, et al[4] made the basic assumption that the turbulent flow pattern is dynamically similar for all Reynolds numbers, provided of course that the jet flow is fully turbulent. Their experimental results appear to justify the assumption. Since the rising column of sewage is a submerged jet with buoyancy, it is basically a combined diffusion and gravitational phenomenon, with the Reynolds number R characterizing the turbulent diffusion and Froude number F, the buoyancy effect.

Re-examination of Rawn-Palmer Data for Reynolds Number Effect.—The original Rawn-Palmer[6] study included 388 experimental observations of the dilution at the surface (S_o) for horizontal jets of fresh water into sea water. The nozzle diameter, D, ranged from $\frac{1}{4}$ in. to 2 in. nominal diameter; various depths (y_o) up to $13\frac{1}{4}$ ft were used. (The experiments were made from a raft in Los Angeles Harbor, and thus simulated an ocean outfall situation, except that the discharge nozzle was not near the bottom as for a real outfall, but was cantilevered down from the raft. However, as previously indicated, this is not considered an important difference.) For each of these runs the values of y_o/D, F, and R were calculated from the original unpublished basic data. Careful analysis of these data by multiple plotting revealed no appreciable influence of Reynolds number which ranged from 5,000 to 40,000.

Reynolds numbers for actual outfalls may easily be of the order of 10^5 or 10^6. For example, a flow at 10 ft per sec from a 1-ft diameter port gives a Reynolds number of approximately 10^6. As the Reynolds number becomes large, its importance as a similarity number markedly decreases for many hydraulic phenomena. It seems reasonable, then, to hypothesize that if S_o is independent of R in the range 5,000 to 40,000 (sometimes a rather critical range) it remains so for larger R-values as well.

Consequently, for the simple case of a rising column in a homogeneous ocean, S_o was considered a function of only two variables, namely

$$S_o = f(y_o/D, F) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

Model studies may be performed on the basis of geometric similarity and Froude's law alone, provided that the flow pattern in the model is fully turbulent.

Graphical Solution for the Dilution.—A graphical solution for S_o was readily formulated, assuming that R is no longer an important variable. Smoothed curves were fitted to the data with the results shown in Figs. 7 and 8. Since both sets of curves represent the same relationship between the variables S_o, y_o/D, and F, values of S may be obtained from either figure.

With results presented in this manner, some interesting observations are possible. For any given horizontal outfall the y_o/D ratio is fixed, and F is directly proportional to V or Q. When y_o/D is about 40 it appears from Fig. 7 that as F increases from 2 up to 5 there is a progressive decrease in S_o. When

[7] "Engineering Hydraulics," ed. by H. Rouse, Ch. II "Hydraulic Similitude" by J. E. Warnock, Wiley, 1950.

F increases beyond 5, S_o remains almost constant up to F = 10, and then starts to increase rapidly as F goes still higher.

When y_o/D is large (say around 80 to 100), then it takes relatively larger Froude numbers to produce the same effects on S_o as those just described for $y_o/D = 40$. The depth becomes a more important factor than F. In other words,

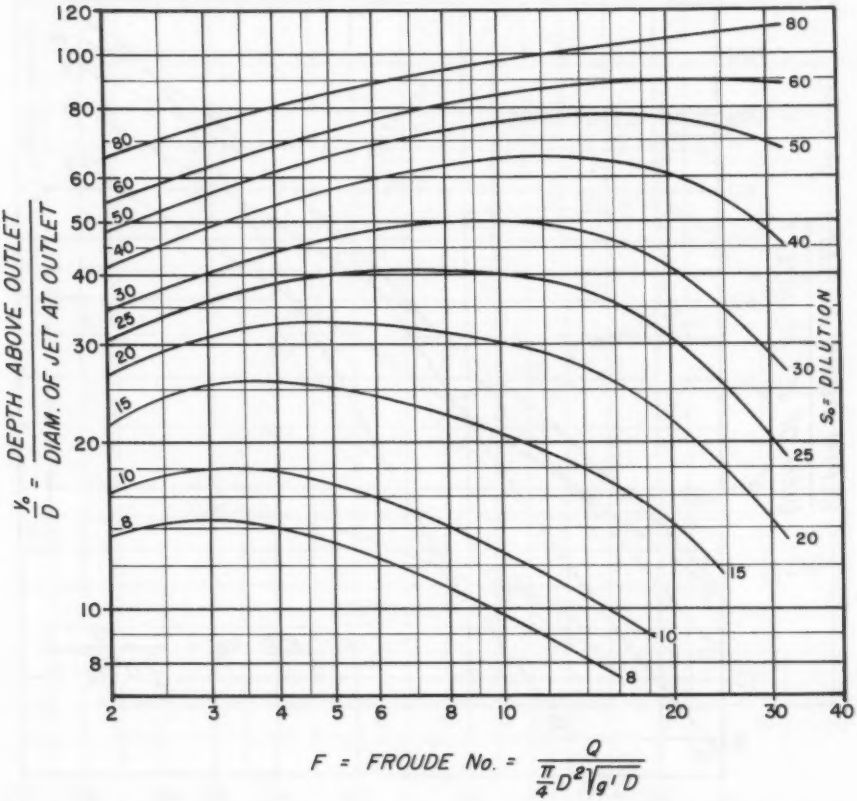

FIG. 7.—DILUTION AS A FUNCTION OF y_o/D AND F FOR HORIZONTAL DISCHARGE

if the column must rise through 100 pipe diameters of depth, increasing the strength of the jetting action—and the horizontal displacement of the jet—for F-values up to 30 or 40, does not result in an increase in S (or a downturn in the constant S-curves in Fig. 7).

The effect on S of changing the several variables in Fig. 7 is shown by the straight lines in Fig. 9. If only the depth y_o is increased keeping all other variables constant, then the change in dilution is found by following a vertical line. On the other hand, an increase in Q for a given outfall (fixed y_o and D) is reflected only in an increase in F in direct proportion with no change in y_o/D. A further possibility is to decrease the size of the outlet, while maintaining the

same depth and same discharge, thereby increasing V; since $F \propto D^{-2.5}$, and $y_o/D \propto (D^{-1})$, the result is a line with slope 2/5 to 1 as shown. Another possibility is that a number of discharge outlets (N) are provided instead of only one, and that outlet diameter D is decreased enough to leave the discharge velocity and aggregate discharge area constant. Since in this case $F \propto D^{-0.5}$, the changed conditions follow the line with 2 to 1 slope.

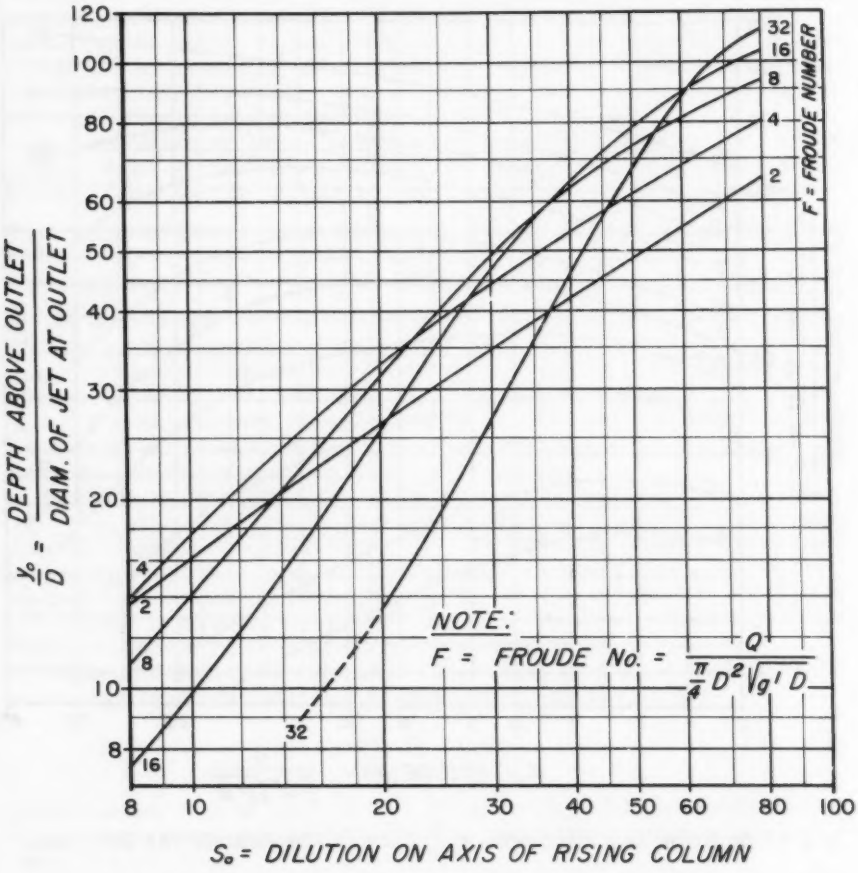

FIG. 8.—DILUTION AS A FUNCTION OF y_o/D FOR CONSTANT F FOR HORIZONTAL DISCHARGE

Certain comparisons can be drawn from Fig. 9 to demonstrate the effectiveness of the several possible methods for increasing initial dilution of sewage effluent in sea water. For example, consider a discharge of 125 mgd through the open end of a 6-ft diameter outfall, lying at 108 ft of depth to pipe centerline; values for the dimensionless variables are $y_o/D = 18$ and $F = 3.0$. These are plotted as point A in Fig. 9. At that point either Fig. 7 or 9 shows that the dilution at the top of the rising column will be $S_o = 10$. Points B, C,

and D in Fig. 9 illustrate three alternative methods for increasing S_o to 40. They may be summarized in Table 3.

The first possibility would involve extending the outfall offshore to a depth of 294 ft; unless increased distance from shore is needed for other reasons, this method of increasing the dilution is so costly as to be infeasible. The third possibility would probably be infeasible in most cases since increasing the velocity of discharge from 6.8 ft per sec to 44 ft per sec results in an increase in velocity head loss from 0.7 ft to 31 ft. The use of 10 or 11 separate

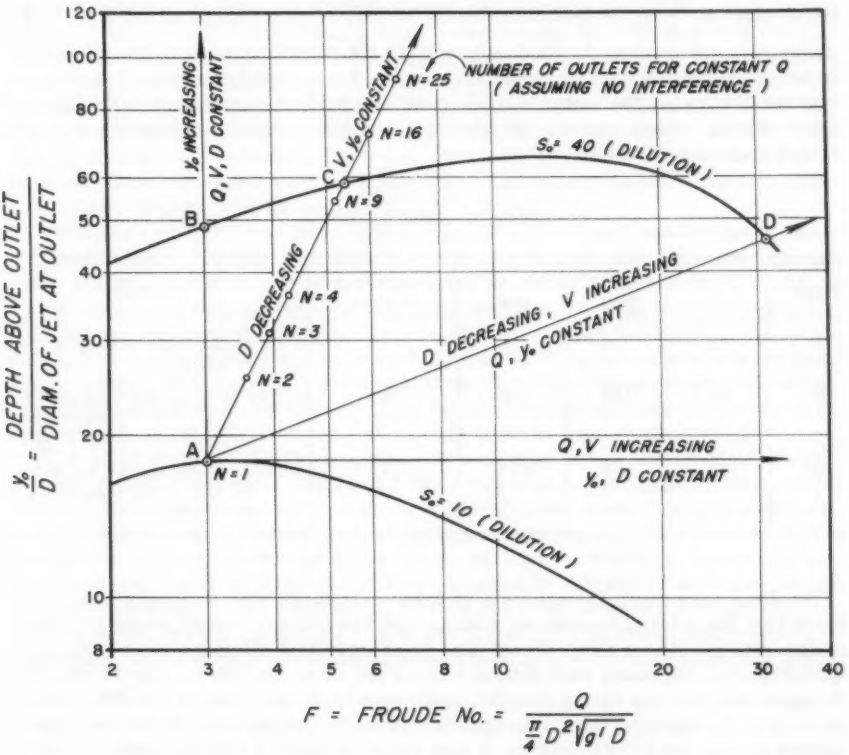

FIG. 9.—DIFFERENT METHODS FOR INCREASING DILUTION

discharge ports adequately spaced to avoid interference appears to be the most feasible alternative. Little additional energy is required, and a diffusion structure can be built at a fraction of the cost required to greatly lengthen or deepen an outfall. Such a line of reasoning led to the decision by engineers of the Sanitation Districts to improve the ocean disposal facilities by means of multiple-outlet diffusers.

Other Investigations.—It has sometimes been suggested that the flow pattern surrounding a submerged ocean outfall is similar to convection over a heat source. Many investigations have been made of convection due to point or line

sources of heat, one of the more recent such being a study by B. R. Morton, G. I. Taylor, and J. S. Turner.[8] When there is a source of heat in a fluid, the fluid adjacent to the source is heated, becomes lighter than the surroundings, and starts rising, thereby inducing convection currents. A rising column of fluid is formed over the heat source although no fluid has been injected into the system. The energy driving this type of flow is entirely derived from the heat of buoyancy. On the other hand, in the case of a submerged discharge from an ocean outfall, the energy available to the flow for entrainment and mixing is of two types: (1) potential energy due to the submergence of sewage in sea water (equivalent to $\frac{\Delta s}{s} y_o$ in hydraulic head), and (2) kinetic energy of the jet. Consequently, the analogy is good only in case the kinetic energy of the discharge is small compared with the potential energy. The method of application of convection theory to the ocean outfall problem will be considered in detail in a later report, where density stratification of the receiving water will also be introduced as a variable.

TABLE 3.—INCREASING S_o

Point	y_o D	y_o, in feet	D, in feet	Number of Outlets	Method
B	49	294	6	1	Increase depth of discharge to 294 ft.
C	58	108	1.86	10.4 (theor.)	Use 10.4 outlets (that is, 10 or 11) of D = 1.86 ft instead of one of D = 6 ft.
D	46	108	2.35	1	Increase discharge velocity by factor of 6.5, using D = 2.35 ft

Another possibility for analysis of a submerged jet with buoyancy is to assume that the dilution increases with L, the curvilinear length along the axis of the rising column (Fig. 6), at the same rate that dilutions in an ordinary submerged jet increase with distance along its axis. In effect, one is simply straightening out the rising column, and neglecting the effect of the difference in density. In essence, then, this approximation is the opposite of the one discussed above; the kinetic energy is now large compared with the potential energy due to density differences.

The second assumption has been used implicitly by E. K. Rice[9] (who discussed only vertical discharges) and by P. Cooley and S. L. Harris.[10] The latter conducted an outstanding experimental investigation under the direction of C. M. White on the use of submerged jets to prevent or destroy density

[8] "Turbulent Gravitational Convection from Maintained and Instantaneous Sources," by B. R. Morton, G. I. Taylor, and J. S. Turner, Proc. Roy. Soc. A., Vol. 234 (Jan-Mar., 1956) pp. 1-23.

[9] "Discharge from Submerged Outfalls," by E. K. Rice, M.S. Thesis, Univ. of Calif., Berkeley, 1949.

[10] "The Prevention of Stratification in Reservoirs," by P. Cooley and S. L. Harris, Jour. of the Inst. of Water Eng., London, Vol. 8, No. 7, Nov. 1954, pp. 517-537.

stratification in a reservoir. Although they were dealing with relatively small density differences, and therefore high Froude numbers, the same principles apply.

Erman A. Pearson[11] has given a comprehensive summary of these and other investigations, and a list of references.

Interference between Jets.—Where there are many ports discharging from a diffuser, there will be some interference between the flow patterns established by the individual jets. If the diffuser ports are arranged in a long line, the flow of diluting water toward the diffuser will be essentially normal to the whole diffuser and thus not radial toward each individual rising plume of effluent. Such a restriction should tend to reduce the dilution somewhat below that which would be obtained from a single port.

Another type of interference is the actual contact of one rising column with another. Rawn and Palmer[6] observed that the diameter of the column at the top is approximately $L_o/3$, where L_o is the curvilinear length as shown in Fig. 6. Hence, when adjacent rising columns have travelled a distance L equal to about three times the port spacing, the columns may be expected to merge and further dilution will be substantially curtailed.

Stratification.—If the receiving water is stratified due to variations in temperature and/or salinity, the flow pattern may be considerably altered. In some instances where warm surface water overlies colder bottom water, a rising column of sewage may not reach the surface, as discussed previously. The sewage effluent discharged from a diffuser mixes with relatively large quantities of cold bottom water, sometimes permitting the mixture to become dense enough that it cannot penetrate a warm surface layer; thus, one of the important advantages of a diffuser may be the creation of a submerged sewage field, which might not occur with discharge from an open-ended outfall.

The problem of predicting possible submergence in the presence of density stratification is complex and cannot be discussed here except briefly. As a first approximation, one could assume an ocean consisting of only two distinct layers—a cold bottom layer and a warm surface layer. For the rise of the sewage in the cold water up to the level of the warm-water interface, the graphical relations in Figs. 7 and 8 can be used to determine an approximate value of the dilution S at that level. If s is the specific gravity of sewage and s_w and s_c the specific gravities the warm and cold layers of sea water respectively, then the mixture will stay submerged below the warm surface layer if

$$\frac{(S - 1)\, s_c + s}{S} > s_w \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

In addition, the density of the mixture should be enough greater than that of the overlying layer to overcome the residual inertia of the rising column.

The relation between dilution and temperature difference may be conveniently illustrated by an example, assuming the following data:

	Temperature, °F	Specific Gravity
Effluent	70°	0.9987
Sea water (dilutent) and salinity = 33,600 ppm.	50°	1.0258

11 "An Investigation of the Efficacy of Submarine Outfall Disposal of Sewage and Sludge," by Erman A. Pearson, State Water Pollution Control Board Publication No. 14, Sacramento, California, 1956.

Consider a column of sewage which rises from a depth of 150 ft to 80 ft, and in so doing becomes diluted with 50 parts of sea water at 50°F. Its specific gravity will then be about 1.0253, which is equivalent to plain sea water at 55° (same salinity, 33,600 ppm). If a thermocline exists above 80 ft, with water significantly warmer than 55°F, then a submerged sewage field may be anticipated. This is commonly the case in the summer and fall months along the southern California coast.

Even if the temperature gradients are not strong enough to prevent the sewage from surfacing, the sewage may still plunge under the surface after leaving the vicinity of the bubble, forming a field which is mostly submerged. This phenomenon was observed in the vicinity of the Whites Point outfalls before the construction of diffusers. Apparently the sewage was heavy enough to sink, but vertical momentum of the large plumes rising from the 5 and 6 ft open-ended pipes could not be overcome by the small density difference.

When the sewage field does stay at the surface, the temperature gradients are still beneficial in making the sewage field thicker, and hence more dilute. Since the edges of a rising jet become diluted most rapidly, this peripheral flow is stopped at a lower level even though the less dilute central core breaks through to the surface. Sampling in the vicinity of the diffuser for the 90-inch outfall at Whites Point indicates that the sewage field frequently extends 50 ft to 100 ft down from the surface when it is not submerged.

Ocean Currents.—Strictly speaking, all of the foregoing material applies to the case of no ocean current. While the aspirator action of the discharging jets from a diffuser is responsible for the intimate mixing of sewage and sea water, nonetheless the complete success of ocean disposal depends on ocean currents for gross exchanges of water. Without these currents the continuous discharge of sewage into the same ocean-water mass would gradually build up an intolerable concentration of sewage, and the sewage discharging from the diffuser would be remixed with older sewage.

If the currents are strong, the dilutions achieved in the ocean may considerably exceed those which may be calculated on the basis of single-port discharges into a calm ocean by Fig. 7. While the initial mixing results from the jet action, whether or not there is a current, the natural turbulence in the ocean may further increase the dilution if there is diluting water available. When a strong current sweeps past a diffuser, the large supply of clean water not only satisfies the demands of the jet entrainment process, but also provides sea water between the small individual diluted sewage streams generated by each port discharge. However, in this case these small individual fields soon mix with the intermediate clean water to form a rather large more or less homogeneous sewage field of a size commensurate with the size of the diffuser. It is this large field which, in the presence of a strong current, will be more dilute even though the initial dilution developed in the rising column itself may not be greatly different from that in a calm ocean.

Considering the total flow of water in a current stream, one may calculate by the continuity principle an average dilution which may be applied to the overall sewage field generated by a diffuser. If Q is the sewage discharge, U is the current velocity, h is the thickness of the sewage field generated, and b is the width of the stream intercepted (that is, the projection of the diffuser normal to the current) then the continuity equation simply states that

$$S_a Q = U\, b\, h \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

Unfortunately there appears to be no reliable way to predict the initial field thickness h in Eq. 8 other than by experience. Probably h will tend to decrease as the current velocity increases (and vice versa), thus somewhat offsetting the increase in S_a with increasing values of U. Nonetheless, unless the density stratification in the ocean is very strong, the field will gradually become thicker as it moves down-current as a result of vertical mixing along the bottom surface of the field and also along the top surface if the field is submerged. Hence an effective value of h can be taken at some distance downstream from the diffuser to allow for some vertical mixing. Values of $\frac{1}{4}$ or $\frac{1}{3}$ of the ocean depth are not considered unreasonable.

As an example of the use of Eq. 8, consider an onshore current of 0.8 ft per sec (0.5 knot) passing the diffuser for the 90-in. outfall at Whites Point (Fig. 2):

$$U = 0.8 \text{ ft per sec}$$

$$b = 2 \times 1200 \times \sin 63^{\circ} 30' = 2150 \text{ ft}$$

$$h = 60 \text{ ft (assumed, but preferably measured)}$$

$$Q = 180 \text{ mgd (average flow)} = 280 \text{ cfs}$$

$$S_a = \frac{0.8 \times 2150 \times 60}{280} = 370$$

It is important to note that this dilution is an average for the entire field, whereas the dilutions represented by Fig. 7 are minimum values, occurring at the center of the individual rising columns, before a large homogeneous field is formed downcurrent.

At this point one may well ask whether the dilution should be calculated in this way or by Fig. 7. If the average dilution for the field as a whole (S_a) is larger than S_o achieved in each rising column with no current, it probably indicates availability of more diluting water than that immediately taken up by the aspirator action. There will be local areas with higher than average concentration in the "bubbles," but as the current carries the field downstream, the natural turbulence in the ocean will cause internal mixing utilizing all the available diluting water and thus promoting uniformity of the field. Hence, in this case the effective dilution may be considered as the average S_a.

On the other hand, if S_o by Fig. 7 is larger than S_s by Eq. 8, then the current does not make additional water available over that drawn in by the initial jet mixing. The actual dilutions achieved over a diffuser will probably be less than S_o by Fig. 7 because of mutual interference between the rising columns (unless the port spacing is very large).

Apart from the beneficial effects of improving the initial dilution at the outfall site, the currents are of course an adverse factor when they carry the sewage field rapidly back to shore. An analysis of the behavior of sewage fields, once formed, is beyond the scope of the present paper, but the problems are briefly described in the following section.

Initial Dilution in Relation to Ultimate Disposal.—The discussion heretofore has dealt almost exclusively with the determination of the initial dilution of sewage with sea water at or near the point of discharge, whether there be single outlets or many. Although it is highly desirable to get as large an initial dilution as possible, that alone does not assure safe disposal. As indicated previously, it is also necessary that the site have favorable currents, and that

the sewage effluent is discharged at a distance from shore sufficient to allow for further reduction of bacterial concentration before the diluted sewage reaches the shoreline.

That dilution alone cannot reduce the concentration of coliform bacteria to the California standard for salt-water bathing (10 per ml) is apparent in that the primary sewage effluent may have concentrations of the order of 10^6 per ml; reduction of concentration by five orders of magnitude is necessary from the time the sewage is discharged into the ocean until it reaches the surf zone in order that shoreline waters are not to exceed 10 coliforms per ml. It is doubtful if any reasonable diffusion system, assisted by further mixing in the ocean as the sewage field travels toward shore, could consistently produce physical dilutions of more than 1,000 to 1. Thus a reduction of bacterial count by two more orders of magnitude must come about through processes such as mortality and sedimentation in the ocean.

In the disposal process the engineer can control only the initial dilution through the choice of outfall site and type of diffuser. Removal of coliforms by natural processes is beyond his direct control; therefore a diffuser which produces a high initial dilution yields greater assurance of good results. Moreover, with a diffuser all other requirements for sewage disposal can be met including turbidity, toxicity, dissolved oxygen levels, odors, and slicks.

An analysis of what takes place in the sewage field after it leaves the vicinity of the diffuser depends on an understanding of the mechanism of turbulent diffusion in the ocean as well as sedimentation and mortality of bacteria. This paper is not broad enough in scope to discuss these factors, but other investigations[12,13,14,15,16] along these lines (primarily in connection with the design of a new ocean outfall for the city of Los Angeles in 1955-56) have been performed.

Summary.—The original Rawn and Palmer[6] experimental data for submerged jets with buoyancy have been reanalyzed to produce Figs. 7 and 8 which give the dilution in the center of rising column of sewage at the ocean surface as a function of two dimensionless numbers: the ratio of depth to jet diameter and the Froude number for the discharge. This new relationship is considered to have more reliability than the earlier empirical equations, inasmuch as the principles of hydraulic models and dimensional analysis have been utilized in developing Figs. 7 and 8.

The most practical way to achieve an increase in initial dilution appears to be to increase the number of discharge jets by means of a relatively long manifold or diffuser. If the receiving water is stably stratified, such as when warm water overlies cold, the small jets of sewage from a diffuser may never appear at the surface, a most desirable objective.

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12 "Studies on Coliform Bacteria Discharged from the Hyperion Outfall," by S. C. Rittenberg, Final Bacteriological Report to Hyperion Engineers by Allan Hancock Foundation, Univ. of So. Calif., Los Angeles, Calif.

13 "Sewage Disposal in Santa Monica Bay, California," by C. G. Gunnerson, Jour. of Sanitary Eng. Div., Proc. ASCE, Vol. 84, No. SA 1, Feb., 1958.

14 "Ocean Outfall Design," Hyperion Engineers (a joint venture), Los Angeles, October 15, 1957.

15 "Methods of Analysis of the Performance of Ocean Outfall Diffusers with Application to the Proposed Hyperion Outfall," by N. H. Brooks, Report to Hyperion Engineers, Pasadena, California, Apr. 5, 1956.

16 "Diffusion of Sewage Effluent in an Ocean Current" by N. H. Brooks, (Paper presented at First International Conference on Waste Disposal in the Marine Environment, July 23, 1959, Berkeley, Calif.; to be published in Proceedings.)

HYDRAULIC DESIGN OF DIFFUSERS

For a given ocean outfall, improvement of dispersal of sewage effluent is accomplished by use of outlet diffusion at the end of the outfall sewer. If the sewage is discharged at a single port or "en masse," its dispersion and dilution will be slower than if it is discharged over a large area through a number of ports. In fact, without the use of multiple-outlet diffusers, other conditions being equal, much longer outfalls into deeper water are necessary to provide the same degree of dispersion and consequent shore protection.

An effective and simple type of diffuser is one which distributes the outflow through many ports over a large area with minimum head loss and interference between rising columns. Diffusers which discharge several radial jets from a single small manifold chamber have been used, but are not as effective because of the interference between rising columns. The following material presumes a diffuser consisting of one long pipe, or several branching ones, with discharge ports at intervals along the pipes.

Basic Hydraulic Requirements.—The successful design of a long diffuser with a large number of outlet ports requires careful hydraulic calculations. The basic hydraulic requirements of such a diffuser are outlined herewith.

Flow Distribution.—The division of the outflow between the various ports should be fairly uniform. If the diffuser is laid on a sloping sea bottom, it will be impossible to achieve uniform distribution between ports for all rates of flow. In such cases, it is advisable to make the distribution fairly uniform at low or medium flow, and let the deeper ports discharge more than the average port discharge during high rates of flow. To allow substantially less than average discharge from deeper ports is considered unsafe from the point of view of possible clogging of the deeper part of the diffuser.

Velocity in Diffuser.—The flow velocity in all parts of the diffuser should be high enough to prevent gross deposition of sludges or grease. For settled sewage, velocities of 2 fps to 3 fps at peak flow are adequate (but borderline) since these will tend to scour material settled during low flow periods. If deposition takes place in any part of the diffuser over an extended period of time, the cross section of the pipe or outlet may become so constricted that locally the velocity will be reduced, a cycle that would accelerate the deposition process. The final result may be complete clogging of the terminal ports and failure of the diffuser to completely perform its dispersal function.

Ease in Cleaning.—Even carefully designed diffusers will require occasional cleaning to remove accumulated grease, slimes, and grit as cited previously under the heading "Outfall System of the Los Angeles County Sanitation Districts." These accumulations tend to increase the apparent friction factor (mainly by decreasing cross-sectional area), thereby reducing flow from offshore ports and increasing flow from inshore ports. Cleaning can be accomplished by flushing or pulling a ball through the line.

Prevention of Sea Water Intrusion.—All ports should flow full in order to prevent the intrusion of sea water into the pipe. Sea water entering the pipe will be stagnant and will tend to trap grit and other settleable matter. Such deposits reduce the hydraulic capacity of the diffuser, thereby limiting its usefulness for future years when higher flows might be expected.

Total Head Loss.—If effluent pumping is necessary or the available gravity head is limited, the total head loss in any proposed diffuser should be kept reasonably small.

Port Design.—The outlet ports may quite satisfactorily be circular holes in the side of the pipe without nozzles or tubes or other projecting fittings. For optimum dilution the jets should discharge horizontally, with no initial upward component of velocity. The inside of the hole should preferably be bell-mouthed to minimize clogging and to provide a discharge coefficient which will remain constant over a period of years.

Hydraulic Analysis.—The hydraulic analysis of a multi-port diffuser is basically a problem in manifold flow and is somewhat complex. A discourse on the subject is not within the purview of this paper, but may be found elsewhere.[17,18] The purpose of the following analysis is primarily to illustrate one manner in which a diffuser can be designed, and to demonstrate some basic principles peculiar to the design of ocean outfall diffusers.

Gravity Effects.—Several gravity effects are important in diffuser flow. The effects of density difference on the rising column has previously been discussed under the heading "Dilution in a Rising Jet of Sewage. Dimensional Analysis" where it was stated that the shape of the discharge jet near the port is governed only by the Froude number F as defined by Eq. 2, assuming ideal flow conditions upstream of the port. According to Rouse,[19] for a circular orifice in a large tank, the Froude number should be greater than 0.59 in order for the orifice to flow full. For a rounded port, it is reasonable to take $F > 1$ as the criterion for flowing full. With every port in the diffuser flowing full, there is no way in which the sea water may re-enter the pipe, once initially expelled, and the diffuser will continue to remain full of sewage effluent.

In making hydraulic calculations, the pertinent pressure at any point in the diffuser is the pressure differential between the fluid inside the diffuser and the sea water outside at the level of the port. Working in reverse order from the deepest or farthest point of a diffuser backward, the decrease in depth tends to increase the pressure differential between the sewage inside the diffuser and the sea water outside, in spite of the fact that the pressure of both sewage and sea water decreases. Henceforth, the use of the terms "pressure" and "pressure head" will herein refer to the pressure differential. The change of this pressure head due to a change of elevation of Δz will be equal to $\frac{\Delta s}{s} \Delta z$.

To the hydraulic engineer designing sewage diffusers for the first time, everything will seem upside down. Indeed, it is quite feasible to visualize the diffuser flow pattern as being inverted, with the vertical scale reduced by the factor $\Delta s/s$ and with the diffuser discharging water into air with the same Froude numbers. The analogy should also help to make clear why it is sometimes difficult to achieve uniformity of discharge from a diffuser built on a sloping ocean floor.

Characteristics of Flow from a Single Port.—The hydraulic analysis of a diffuser is essentially a step-wise process starting at the extreme outer end. The ports are assumed to be far enough apart so that the flow in the vicinity of any one port is independent of the rest of the diffuser flow. This is a reasonable assumption, as the port spacing in an adequate diffuser would be at least 10 port-diameters, and probably considerably more. The discharge from

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[17] "Mechanics of Manifold Flow," by J. S. McNown, Trans. ASCE, Vol. 119, (1954) p. 1111.

[18] "Application of Conformal Mapping to Divided Flow," by J. S. McNown and E. Y. Hsu, Proc. First Midwestern Conf. on Fluid Dynamics, J. W. Edwards, Ann Arbor, Mich., 1951, pp. 143-155.

[19] "Elementary Mechanics of Fluids," by H. Rouse, Wiley (1946), p. 105.

each port is figured separately in turn, and added to the quantity of flow carried by the diffuser pipe downstream. Between consecutive ports, the effective pressure head is increased by the amount of the friction loss and the "density head" ($\frac{\Delta s}{s}\Delta z$). The key to the problem is the analysis of lateral discharge from a port in the side of a pipe.

The rate of discharge, q, from an orifice or port in the side of a pipe is expressed by

$$q = C_D\, a\sqrt{2\,g\,E} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7)$$

in which C_D is the discharge coefficient, a denotes the cross-sectional area of port, and E is the total head in the main flow at the port. The total head, E, includes the pressure head in the main pipe relative to the ocean at the location of the port plus the velocity head of the main flow. In the neighborhood of the discharge port, it is assumed that there is no energy loss for the main flow in passing the port. In other words, there is perfect pressure recovery compensating for reduction in velocity head in the main flow because of the lateral discharge. McNown[17] has shown this to be a good assumption.

The discharge coefficient, C_D, is not a constant along the diffuser, but decreases as the velocity head ($V^2/2\,g$) becomes a larger part of the total energy (E). By an extension of a theoretical analysis of branching flow by McNown and Hsu[18] it has been found that C_D can be expressed as a function of the ratio $\frac{V^2}{2\,g}/E$ as shown in Fig. 10. Theoretically the curve in Fig. 10 applies only to small discharges from small holes with diameter less than one-fourth of the main pipe diameters, as is always the case with diffusers. For small q, the velocities upstream and downstream of the port are approximately equal (that is, $V_n \approx V_{n-1}$) and either V_n or V_{n-1} may be used to calculate the ratio $\frac{V^2}{2\,g}E$, although V_{n-1} is the more convenient. Furthermore, it is presumed that the discharge coefficient for negligible velocity of approach ($\frac{V^2}{2\,g}/E \to 0$) is 0.61 for a sharp-edged port and 0.91 for a rounded port. These values are reasonable for discharge from a tank or reservoir.

Calculation Procedure.—The whole calculation procedure as used in the design of a diffuser may be formulated mathematically as follows:

Let: D = diameter of pipe;

d_n = diameter of nth port, counting from offshore end;

a_n = area of nth port;

V_n = mean pipe velocity between nth port and $(n+1)^{th}$ port (see Fig. 10);

$\Delta V_n = V_n - V_{n-1}$ = increment of velocity due to discharge from nth port (or group of ports);

$h_n = \Delta P_n/\gamma$ = difference in pressure head between the inside and the outside of the diffuser pipe just upstream of nth port (expressed in feet of sewage);

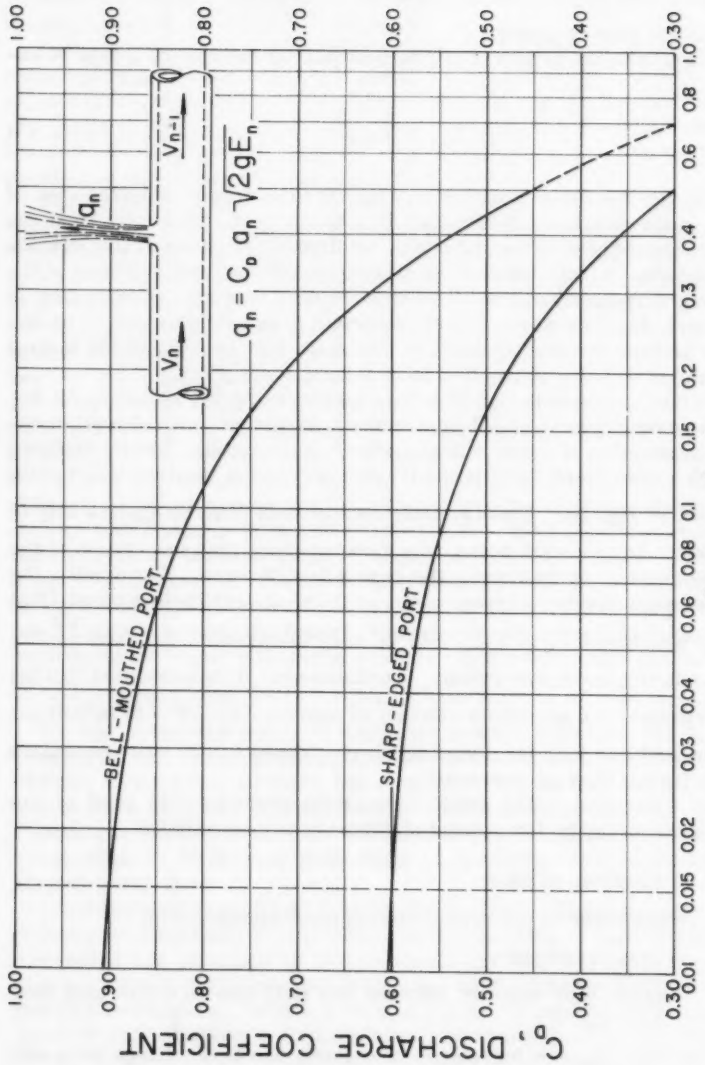

FIG. 10.—DISCHARGE COEFFICIENT FOR A SMALL PORT ON THE SIDE OF A PIPE

$E_n = h_n + \frac{V_n^2}{2g}$ = total head at nth port (same either side by assumption above);

C_D = discharge coefficient for ports (see Fig. 10);

q_n = discharge from the nth port;

h_{fn} = head loss due to friction between (n + 1) and nth port;

L_n = distance between (n + 1) and nth port;

f = Darcy friction factor;

Δz_n = change in elevation between (n+1) and nth port (measured to center of port; positive when (n+1) port is not as deep as the nth port);

s = specific gravity of sewage;

Δs = difference in specific gravity between sea water and sewage

First it is necessary to select E_1; then q for the first port is:

$$q_1 = C_D a_1 \sqrt{2 g E_1} = C_D \frac{\pi}{4} d_1^2 \sqrt{2 g E_1} \quad \ldots\ldots\ldots \quad (8)$$

Next, one finds the pipe velocity

$$V_1 = \Delta V_1 = \frac{q_1}{\frac{\pi}{4} D^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (9)$$

and velocity head $V_1^2 / 2g$.

Proceeding to port No. 2, one finds E_2 by

$$E_z = E_1 + h_{fi} + \frac{\Delta s}{s} \Delta z_1 \quad \ldots\ldots\ldots\ldots\ldots \quad (10)$$

The ratio $\frac{V_1^2}{2g} / E_2$ is calculated for use in Fig. 10 to find C_D.

Then

$$q_2 = C_D a_2 \sqrt{2 g E_2} \quad \ldots\ldots\ldots\ldots\ldots \quad (11)$$

and

$$V_2 = V_1 + \Delta V_2 = V_1 + \frac{q_2}{\frac{\pi}{4} D^2} \quad \ldots\ldots\ldots\ldots\ldots \quad (12)$$

This procedure is continued step by step back up the diffuser using the general relations;

$$q_n = C_D\, a_n \sqrt{2\, g\, E_n} \quad \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

$$C_D = \phi \left(\frac{V_{n-1}^2}{2\, g} \Big/ E_n \right) \quad \ldots\ldots\ldots\ldots\ldots (14)$$

$$\Delta V_n = \frac{q_n}{\frac{\pi}{4} D^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

$$V_n = V_{n-1} + \Delta V_n \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

$$E_{n+1} = E_n + h_{fn} + \frac{\Delta s}{s} \Delta z_n \quad \ldots\ldots\ldots\ldots\ldots (17)$$

$$h_{fn} = f \frac{L_n}{D} \frac{V_n^2}{2\, g} \quad \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

A tabular form was devised to facilitate the calculations as illustrated by Table 4. If the port discharges and pipe velocities change slowly, it is expedient to make the stepwise calculations for small groups of ports, as was done in Table 4. In this case, Eq. 15 is changed to read

$$\Delta V_n = m \frac{q_n}{\frac{\pi}{4} D^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (19)$$

wherein m is the number of ports considered in a group.

By the nature of the calculations, it is apparent that one cannot decide on a particular total flow before starting the calculations. It is necessary to estimate the flow from the end port (q_1) which will correspond to the desired total flow.

Selection of Port Sizes and Spacing and Pipe Sizes.—During the process of the calculation, the designer is at liberty to change the pipe size, the port size, and/or the port spacing. To keep the velocity high enough at the end of the diffuser, it is sometimes necessary to reduce the size of the pipe in one or more steps from beginning to end of the diffuser. The size of the discharge ports may be varied in order to keep the discharge uniform from port to port. The spacing between ports is rather inflexible, inasmuch as practical considerations dictate that the spacing be either equivalent to the length of a pipe section or multiple or simple fraction thereof. The entire design process inevitably requires some trial and error arrangements in order to get one arrangement which is satisfactory at various total rates of flow.

For a diffuser which is laid on a zero slope, the relative distribution of flow would be the same at all rates of discharge. This is because all the head terms are proportional to the square of the velocity. In that case, where there are no

TABLE 4.--SAMPLE HYDRAULIC CALCULATIONS FOR DIFFUSER DESIGN, 90 - IN. OUTFALL DIFFUSER, FLOW OF 79 MGD

Port Number	Distance from End, in Feet[a]	Port Diameter, d_n, in Inches	Port Area, a_n, in Square Feet	Pipe Diameter, D, in Feet	Total Head, E_n, in Feet, From Col. 18	Ratio, Col. 12 ÷ Col. 6	Discharge Coefficient, C_D, from Fig. 10	Port Discharge, in cfs, q_n = Col. 8 x Col. 4 x $\sqrt{2g}$ Col. 6	Increment in Pipe Velocity, in fps, $\Delta V = \frac{m}{a}$ Col. 9	Pipe Velocity, V_n, in fps	Velocity Head, in Feet	Friction Factor, f	Distance to Next Port, L_n, in Feet	Head Loss, in Feet, h_{fn} = Col. 12 x Col. 13 x Col. 14 ÷ Col. 5	Decrease in Depth, Δz_n, in Feet	Density Head, $\frac{\Delta s}{s} \Delta z_n$, in Feet	Total Head, in Feet, E_n = Col. 6 + Col. 15 + Col. 17	Remarks $\Delta s/s$ = 0.026, A = $\pi D^2/4$ = 19.63 sq ft, m = number of ports in group in Col. 1
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)
1	0	15.0	1.227	5.00	0.199	0	0.91	4.00	0.204	0.20	0.00065	0.024	24	0.000+	0.43	0.011	0.210	Slope of pipe = 1.8% for first
2-3	24	7.5	0.307	"	0.210	0.003	0.91	1.03	0.105	0.31	0.0015	"	48	0.000+	0.86	0.022	0.232	504 ft
4-5	72	"	"	"	0.232	0.006	0.91	1.08	0.110	0.42	0.0027	"	"	0.001	"	"	0.255	f = 0.024 is equivalent to
6-7	120	"	"	"	0.255	0.011	0.905	1.13	0.115	0.53	0.0044	"	"	0.001	"	"	0.278	Manning n = 0.015
8-9	168	"	"	"	0.278	0.016	0.90	1.17	0.119	0.65	0.0066	"	"	0.002	"	"	0.302	
10-11	216	"	"	"	0.302	0.022	0.89	1.21	0.123	0.77	0.0093	"	"	0.002	"	"	0.326	
12-13	264	"	"	"	0.326	0.029	0.885	1.25	0.127	0.90	0.013	"	"	0.003	"	"	0.351	
14-15	312	"	"	"	0.351	0.036	0.875	1.28	0.130	1.03	0.016	"	"	0.004	"	"	0.377	
16-17	360	"	"	"	0.377	0.043	0.87	1.32	0.134	1.16	0.021	"	"	0.005	"	"	0.404	
18-19	408	6.5	0.230	"	0.404	0.052	0.86	1.01	0.103	1.26	0.025	"	"	0.006	"	"	0.432	
20-21	456	"	"	"	0.432	0.057	0.855	1.04	0.106	1.37	0.029	"	"	0.007	"	"	0.461	
22-23	504	"	"	"	0.461	0.063	0.85	1.07	0.109	1.48	0.034	"	"	0.008	0.34	0.009	0.478	Slope of pipe = 0.7% for
24-25	552	"	"	"	0.478	0.071	0.84	1.07	0.109	1.59	0.039	"	"	0.009	"	"	0.496	remaining 696 ft.
26-27	600	"	"	"	0.496	0.079	0.835	1.08	0.110	1.70	0.045	"	"	0.010	"	"	0.515	
28-29	648	"	"	"	0.515	0.087	0.825	1.09	0.111	1.81	0.051	"	"	0.012	"	"	0.536	
30-31	696	"	"	"	0.536	0.095	0.82	1.11	0.113	1.92	0.057	"	"	0.013	"	"	0.558	
32-33	744	"	"	"	0.558	0.102	0.81	1.12	0.114	2.03	0.064	"	"	0.015	"	"	0.582	
34-35	792	"	"	"	0.582	0.110	0.80	1.13	0.115	2.15	0.072	"	"	0.017	"	"	0.608	
36-37	840	"	"	"	0.608	0.118	0.795	1.14	0.116	2.27	0.080	"	"	0.018	"	"	0.635	
38-39	888	"	"	"	0.635	0.126	0.785	1.16	0.118	2.39	0.089	"	"	0.020	"	"	0.664	
40-41	936	"	"	"	0.664	0.134	0.775	1.17	0.119	2.51	0.098	"	"	0.022	"	"	0.695	
42-43	984	"	"	"	0.695	0.141	0.77	1.19	0.121	2.63	0.107	"	"	0.025	"	"	0.729	
44-45	1032	"	"	"	0.729	0.147	0.765	1.21	0.123	2.75	0.117	"	"	0.027	"	"	0.765	
46-47	1080	"	"	"	0.765	0.153	0.755	1.22	0.124	2.87	0.128	"	"	0.029	"	"	0.803	
48-49	1128	"	"	"	0.803	0.159	0.75	1.24	0.127	3.00	0.140	"	"	0.032	"	"	0.844	
50-51	1176	"	"	"	0.844	0.166	0.745	1.27	0.129	3.13	0.152	"	24	0.018	0.17	0.004	0.866	Wye is 1200 ft. from end
Sum			13.95					Q=61.5 each log					1200	0.31	14	0.36		Q_{TOTAL} = 123cfs = 79 mgd

[a] When a group of ports is used, distances are to the centroid of the group.
[b] Aggregate port area for both legs is 27.9 ft.^{2} or 63% of cross section of 90-inch outfall pipe.

differential elevations, one calculation would suffice for all rates of flow. For example, to double the rate of flow, one would need only to quadruple all the heads and double all the velocities and discharges.

It is essential that the end of the diffuser pipes be bulkheaded, otherwise the flow will not be forced out of the discharge ports near the end of the diffuser, and an excess of flow will be discharged through the open-ended pipe. The bulkheads should be removable for flushing the line.

In the process of making the hydraulic calculations it was found that a good rule of thumb was to assure that the sum of all the port areas is less than the cross-sectional area of the outfall pipe. In fact, it was quite apparent that it is impossible to make a diffuser flow full if the aggregate jet areas exceeds the pipe cross-section area, since that would mean that the average velocity of discharge would have to be less than the velocity of flow in the pipe and that the flow would have to be decelerated before being discharged from the ports. This is not physically possible; consequently some of the ports would not flow full, or not flow at all, and the diffuser would not work properly. It is probable that the principal reason for the difficulties of some of the early multiple-outlet diffusers, such as the Deer Island Outfall[4] in Boston, Mass. was the failure to observe that simple criterion.

Application to Outfalls of the Los Angeles County Sanitation Districts.—The foregoing calculation procedure is illustrated by a set of design calculations for the diffuser for the 90-in. outfall (see Fig. 2). Table 4 lists hydraulic calculations for a low flow of 79 mgd, a condition selected as an example because the density head terms $\frac{\Delta s}{s} \Delta z_n$ have a significant effect on the results. In the following material, the several steps in the hydraulic design of the diffuser for the 90-in. outfall will be reviewed.

Initially, it was necessary to select the location, orientation, and length of the two diffuser legs from oceanographic and economic considerations, including initial dilutions obtainable, prevailing currents, temperature profiles, and costs of alternatives. It was decided to make two diffuser legs 1,200 ft long with an included angle of 127°, the bisector of the angle being approximately normal to the coastline. In this way, the width of the band of current intercepted (or the width of sewage field formed) was largest for a shoreward current, and least for a longshore current. No ports were provided in the main trunk, as was done in the 72-in. outfall, since the width of the sewage field would thereby only be increased for longshore currents which are not critical. Furthermore, the grade of the pipe is much steeper on the main trunk making a balanced hydraulic design more difficult to achieve if ports were to be placed in the main pipe.

The two diffuser legs consist of 60-in. reinforced concrete pipe with one port in each 24-ft section, facing alternate directions. The two legs were laid out with as nearly identical profiles as possible, to ensure symmetry and equal distribution of the flow.

From the detailed hydraulic calculations, the designer is able to select reasonable port and pipe diameters. In the case in question it was found that the diffuser pipes could satisfactorily be of a single diameter throughout—60 in. After some trial and error calculations, a port size of $7\frac{1}{2}$ in. was chosen for the outermost 14 sections, and $6\frac{1}{2}$ in. for the other 34 sections in each leg.

Quadruple-sized ports (15-in. diameter) were placed in the center of each of the end bulkheads in order to maintain favorable pipe velocity and to make use of the better than average diffusion opportunity at the extreme borders of the sewage field formed by the diffuser.

In Table 4, the Δz_n-values are based on an average measured grade of 1.8% for the outermost 500 ft and 0.7% for the inner 700 ft, thus accounting for a total drop of about 14 ft. With a 2.6% density difference, this drop makes a total head differential of 0.36 ft. of fresh water head; that is, even under hydrostatic conditions the pressure head difference between inside and outside of the pipe would be 0.36 ft greater at the wye than at the end. From the top of Col. 6 and the bottom of Col. 18 in Table 4 it may be noted that the total head increases from 0.20 ft to 0.87 ft as one works backward from the end of the wye. Of this increase of 0.67 ft, 0.36 ft is due to change in elevation and only 0.31 ft to friction. Consequently, this set of calculations demonstrates the importance of considering the differential elevation at low flow. At a flow of 240 mgd, the computed total head was found to rise from 3.25 ft. to 6.67 ft, an increase of 3.42 ft; of this amount 3.06 ft is for friction loss, with only 0.36 ft for change in elevation, as before.

The effect of the sloping bottom on the distribution of flow between the ports is apparent in Fig. 11 on which are plotted the distributions for a high flow of 240 mgd and a low flow of 79 mgd. The port sizes have been selected in a way which makes the port discharges practically uniform at the low flow, but at high flow, the offshore ports discharge on the average about 25% more than the inshore ports. Going shoreward from the end of the diffuser (that is, in the order in which the calculations are made), the flow from each port gradually increases for low flow primarily because of the increase in elevation. On the other hand, at high flow, the situation is reversed; in spite of the increasing elevation and accumulated pipe friction losses, the port discharge decreases very gradually because the discharge coefficient decreases faster than the head increases. It is seen that there are four factors which tend to change the port discharge: change in elevation, pipe friction, change in C_D, and change in port diameter. For a successful diffuser these four factors should be kept as nearly in balance as possible.

As indicated previously, an hydraulic design yielding discharge distributions such as those in Fig. 11 is safe inasmuch as there is virtually no danger of gradual choking of the offshore end of the diffuser. If the friction factor f became considerably higher than the design value of 0.024 (which is equivalent to Manning n = 0.015), the discharges from offshore ports would be decreased while the discharges from nearer-shore ports would be increased. Consequently, as the diffuser is designed, an increase in f would only tend to make the distribution of discharge more uniform at high rates of flow.

The velocity distribution within the diffuser pipe may be read from Table 4, Col. 11. At low flow (79 mgd), the velocity in the pipe has a minimum of 0.20 fps at the very end, and maximum of 3.12 fps just downstream from the wye. At the high flow (240 mgd), the corresponding velocity figures are 0.82 and 9.46 fps, respectively. Experience has shown that cleaning is necessary every few years, especially in the last few sections of pipe where the velocity is the lowest. This has been easily accomplished by opening the end bulkheads and flushing with as much flow as possible by diverting effluent from the other outfalls. Hydraulic calculations for the flushing operation can also be made by the procedure outlined previously. With the bulkhead opened, a large part of

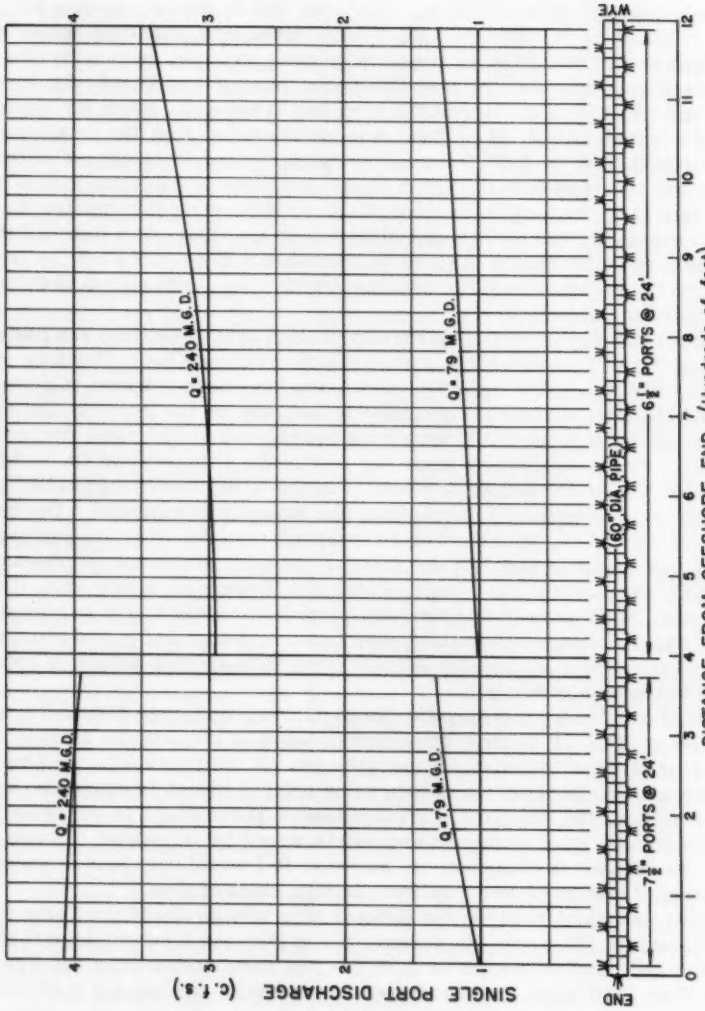

FIG. 11.—CALCULATED DISTRIBUTION OF DISCHARGE ALONG EACH LEG OF DIFFUSER FOR 90-IN. OCEAN OUTFALL

the flow discharges from the open end and pressure within the manifold is reduced; near the end, the pressure differential will be so small that the ports will discharge very little, while further back up the line the friction loss and density head will put the pipe again under positive differential pressure resulting in substantial port discharges, although much less than during ordinary operation at the same total flow.

The lowest value of the Froude number under the low flow condition is 3.2 (by Eq. 2) at the port in the end bulkhead. Since this is considerably greater than 1, the diffuser must operate full of sewage at all times, with no encroachment of the sea water in the pipe.

The arrangement of the columns in Table 4 is general and could be applied to any diffuser problem. Full columns have been allotted to D and f (Cols. 5 and 13), even though they are constant throughout this example, to indicate that it is simple to make changes in D and f if desired. If the pipe diameter D is changed, it is necessary to change V also (Col. 11) and add an energy loss for the transition.

The diffusers for the 72-in. and 60-in. outfalls were designed earlier by a similar procedure, which differed only in some details. The main characteristics of all three diffusers are summarized in Table 2 and the layouts are shown in Figs. 1 & 2. It may be noted that the ports for the 72-in. outfall diffuser are rounded, while those for the 60-in. are not. In the former, the holes were cast in new pipe, whereas in the latter the holes were drilled into the existing pipe while in place on the ocean floor. Hydraulically speaking, it makes little difference whether the ports are rounded or not; one simply has to use a larger sharp-edged hole to be equivalent to the smaller, rounded port. However, for a sharp-edged port through the wall of a heavy concrete pipe, there is some doubt as to whether it will function as an orifice or a short tube, and thus it is more difficult to predict the discharge coefficient.

The design calculations for the 72-in. outfall diffuser were checked by some crude field measurements of port discharge velocity using a specially rigged Price current meter held in front of the ports by a diver with the meter read from the boat. In Fig. 12 measured port discharges for a total flow of 53 mgd are compared with the calculated discharges for flows of 42, 89, and 138 mgd. The agreement is seen to be reasonable except for the first port on the upstream leg which may well be an error in measurement.

Summary.—A diffuser can be designed by calculating the port discharges one at a time starting with the offshore end. For discharge through lateral ports in a pipe, the discharge coefficient is a variable function of the ratio of the velocity head to the total head in the main pipe as shown by Fig. 10. Balanced distribution of discharge among the ports can be secured by varying the port diameter. A necessary requirement in selecting port sizes is to keep the sum of port areas less than the cross-sectional area of the outfall.

In Table 4 the procedure for design is illustrated for the diffuser for the 90-in. diameter outfall of the Los Angeles County Sanitation Districts. Although field confirmation of the hydraulic design is difficult, limited observations confirm the theory; the three outfall diffusers in use at Whites Point have given satisfactory service for several years.

CONCLUSIONS

1. The efficiency of disposal of primary sewage effluent to the ocean can be greatly improved by a good dispersal system.

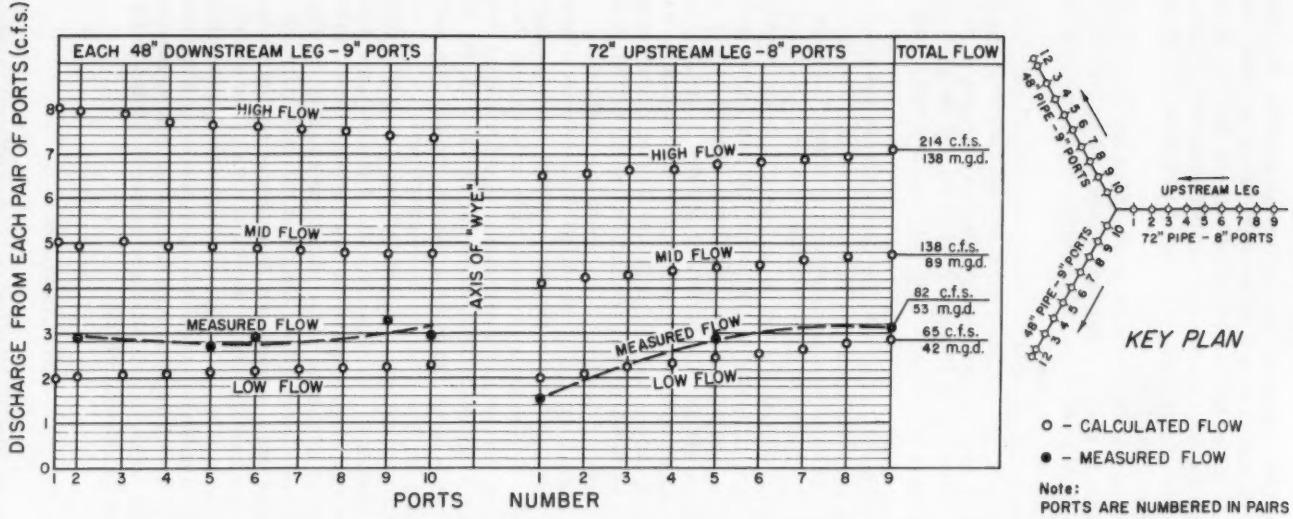

FIG. 12.—PREDICTED AND MEASURED DISCHARGES FROM 72-IN. OCEAN OUTFALL DIFFUSER

2. Increased dispersal or dilution of the sewage in sea water can most easily be achieved by a large multiple-port manifold or diffuser. By discharging the sewage flow in a large number of small jets instead of a single jet from an open-ended pipe, larger dilutions of the sewage in sea water will be achieved in the rising column.

3. In the presence of stable thermal gradients in the ocean the rising plume of sewage may never reach the surface. This will occur when the sewage mixes with sufficient cold bottom water to make the mixture density greater than that of the overlying surface layer of warmer sea water.

4. The possibility of achieving a submerged sewage field when thermal gradients exist is greatly enhanced by the use of a large multiport diffuser.

5. Ocean currents are necessary for satisfactory disposal to effect exchanges of the water masses.

6. Strong ocean currents passing over a diffuser may also give greatly increased dilution in the sewage field formed by the flow from the diffuser. The arms of the diffuser should be so oriented as to intercept a wide band of the current when its direction is unfavorable from the point of view of shoreline pollution.

7. The Los Angeles County Sanitation Districts have installed three large multiple-port diffusers which have performed satisfactorily in accordance with the principles enunciated herein. (For layout, see Figs. 1 and 2.)

8. The operation of the diffuser has resulted in reduced coliform bacteria populations along the shoreline in spite of the steadily increasing flow of sewage. (See Fig. 3)

9. There has been great improvement in the appearance in the water surrounding the outfall and a large increase in physical dilution. (See Table 2)

10. Two of the three diffusers have been cleaned without difficulty after 5 and 2 yr of service respectively; the third diffuser did not require cleaning, as it has not been in constant use.

11. The original Rawn and Palmer data[6] for dilution in a rising column from a horizontal discharge have been reanalyzed, resulting in the graphical presentations of Figs. 7 and 8. The dilution in the center of the rising column is now expressed as a function of the Froude number F and the ratio of depth to jet diameter, y_o/D.

12. A study of these relationships shows that the most feasible way to increase the dilution is to reduce the jet diameter D by use of a large number of ports, without necessarily increasing the velocity of discharge.

13. Large diffuser manifolds should be so designed that (a) the distribution of flow between ports is fairly uniform; (b) the velocities within the diffuser pipes are sufficient to avoid undue deposition; (c) the structure is easily cleanable; (d) the system operates full of sewage without sea-water intrusion; (e) the total additional head loss is reasonable; and (f) the ports themselves are of a simple design.

14. For a diffuser constructed of large pipes with small holes along the sides the hydraulic analysis involves a variable coefficient of discharge, depending on the ratio of the velocity head to total head within the diffuser (Fig. 10). Calculations of the head-discharge characteristics for the system must start at the downstream end and proceed upstream.

15. A successful diffuser usually has an aggregate port area which is less than the outfall cross-section area.

The scope of this paper has been limited to a discussion of initial dilution and hydraulic design of diffusers together with verification and illustrations taken from the history of the outfall system of the Los Angeles County Sanitation Districts. No attempt has been made to discuss in detail the complex phenomena which take place within the sewage field once it is formed. Nonetheless, considerations of factors such as coliform mortality and sedimentation, and diffusion by natural turbulence in the ocean should be considered in any comprehensive outfall design. These subjects have been treated elsewhere.[12,13,14,15,16]

APPENDIX.—NOTATION

a_n = area of nth port;

b = initial width of sewage field;

C_D = discharge coefficient for ports (see Fig. 10);

D = diameter of sewage jet at point of discharge or, diameter of diffuser pipe;

d_n = diameter of nth port, counting from offshore end;

$E_n = h_n + \frac{V_n^2}{2g}$ = total head at nth port (same either side by assumption);

$F = \frac{V}{\sqrt{g' D}}$ = Froude number;

f = Darcy friction factor;

g = acceleration due to gravity;

$g' = g\frac{\Delta s}{s}$ = apparent acceleration due to gravity;

h = initial thickness of sewage field;

h_n = difference in pressure head between the inside and the outside of the diffuser pipe just upstream of nth port (expressed in feet of sewage);

h_{fn} = head loss due to friction between (n + 1) and nth port;

L_o = distance along axis of rising column from point of discharge to water surface (see Fig. 6);

L_n = distance between (n + 1) and nth port;

p = fraction of sewage (by volume) in a sewage-sea water mixture;

Q = rate of sewage discharge;

q_n = discharge from the nth port;

$R = \frac{V D}{\nu}$ = Reynolds number;

S = $1/p$ = dilution ratio at a point on the axis of a rising plume of sewage;

S_o = dilution S at water surface ($y = y_o$);

S_a = $U b h/Q$ = average dilution in sewage field;

s = specific gravity of sewage;

Δs = difference in specific gravities of sea water and sewage;

U = velocity of ocean current;

V = jet velocity;

V_n = mean pipe velocity between nth port and $(n + 1)^{th}$ port;

ΔV_n = $V_n - V_{n-1}$ = increment of velocity due to discharge from nth port (or group of ports);

y = height above center of outlet;

y_o = total depth from center of outlet to water surface;

Δz_n = change in elevation between $(n + 1)$ and nth port (measured to center of port; positive when $(n + 1)$ port is not as deep as the nth port); and

ν = kinematic viscosity of sewage

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SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

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THE HYDROLOGY OF URBAN RUNOFF[a]

Closures by A. L. Tholin and Clint J. Keifer

A. L. THOLIN,[1] F. ASCE.—Since the preparation of the subject paper, the writer has assumed administrative duties in the general field of public works and, as a consequence, has been less intimately associated with the design of sewers, although such work is still within the writer's administrative assignments. The co-author, Mr. Keifer, however, has continued in a lead position in the hydraulic design of new sewer systems for Chicago and has continued some basic studies of the rainfall-runoff relationship and the application of the electronic computer to the "Chicago Hydrograph Method," as outlined in the paper.

In view of the partial separation of the present daily duties of the authors, it appears to be reasonable to prepare separate closing discussions. The writer will limit his discussion to general concepts. Mr. Keifer will cite some specific results obtained from a gauging station established since the original presentation of the paper and indicate the results of additional computations programmed for and performed with the aid of an electronic computer, also made available to the Sewer Planning Division since that time. He will also discuss other specific matters raised by the discussions.

The writer appreciates the high degree of competence represented by those who have contributed to the discussion. It is hoped that other competent writers may be stimulated to present independent papers on the subject in the near future.

Since the height and shape of the intensity-duration-frequency curve of anticipated rainfall is basic to the development of the Chicago Hydrograph Method, the comments on this subject by Mr. Wolf are very pertinent and will stimulate the Sewer Planning Division to reappraise the correctness of the Eltinge-Towne formula. This formula was statistically derived from data collected over a 42-yr period, prior to 1942. A study of other storms of more recent record may indicate, as Mr. Wolf suggests, that the values of rainfall intensity are too low for the storms of longer duration. An upward adjustment of these values for the longer rainfall periods would affect computed runoff rates for the larger drainage areas. In Chicago, there are, at present, four such areas delivering runoff to stormwater pumping stations. Another such area and pumping station is contemplated. A current reappraisal, prior to the design of the latter, is in order.

The writer wishes to point out, however, that as shown in Fig. 29 of Mr. Wolf's discussion the values obtained from the Eltinge-Towne formula are in close agreement with his suggested revision for all precipitation periods less

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[a] March, 1959, by A. L. Tholin and Clint J. Keifer.

[1] Administrative Engr. of Pub. Works, Dept. of Pub. Works, Chicago, Ill.

than 60 min. Most of the drainage areas in Chicago have "time of concentration" values of less than 60 min, and sewers designed for these shorter periods of rainfall will not be overloaded by the attenuation of the rainfall hyetograph at rates susceptibly higher than those now assumed.

With regard to rainfall rates used in the design storm pattern, Messrs. McPherson and Willis correctly point out that in the illustrative problems, the authors disregard the reducing effect of less than "point rainfall" as an average rate for drainage areas of less than 640 acres. This is purely arbitrary, for the sake of simplification of computations for the larger number of smaller areas. The application of the reduction factor "K" in Table VII makes a sudden downward adjustment of 11% in the estimated peak flows when the cumulative drainage area reaches 640 acres. Perhaps this adjustment should take place at a smaller cumulative area. McPherson and Willis also inquire about the meaning of "duration" in Fig. 24. The terminology on this chart should have been "Storm of 3 Hour Duration or Maximum 3 Hours of Longer Storm," etc. The curves were derived from records of twenty-two major rainstorms occurring over a 12-yr period.

Mr. Bock raises a question with regard to the validity of the value of "r" as being equal to 3/8, for the establishment of the chronological location of the peak rate of rainfall with regard to the total duration of the "design storm." The derivation of this value was more thoroughly discussed in a previous paper by Messrs. Keifer and Chu.(1) The value derived agrees closely with the value suggested for summer thunderstorms in St. Louis in the paper by Messrs. Horner and Jens.(4) No assumed design storm is ever reproduced in nature, but the authors still believe that the assumed location of the peak rate is a reasonable assumption for design purposes in Chicago.

Mr. Bock correctly states that "the improvement and verification of any method for urban storm drainage design rests ultimately on measurements of rainfall-runoff on urban areas." His work in Baltimore is a notable contribution of factual data in this regard. For proper application of such factual data, however, to drainage areas other than the one gauged, it is necessary to know not only what the rates of runoff were but why they were so. No two drainage areas have identical physical characteristics. If one or more of the variable characteristics of an ungauged area are different from those in the gauged area, how should one adjust the data? Such a question leads back to the analytical methods dealt with in the paper.

The writer believes that gaugings by themselves, as a basis for design must be far more voluminous than presently available and all the physical features of the drainage area must be considered in the appraisal and use of gauging data. Gaugings, on the other hand, performed for verification or modification of rigid methods of analysis and synthesis of ascertainable physical characteristics and their effect upon runoff, can establish the validity of the analytical methods so that in the future, such gaugings are no longer necessary. Since they are expensive and have limited direct applicability to non-identical areas, gaugings for verification of basic analytical methods are, in the long run, more economical than gaugings for direct use in design.

Messrs. Izzard and Armentrout state, with regard to the work of the authors, that "while this work is a definite contribution to the field of urban drainage, it must be examined for verification and simplification of the procedure." To this both authors agree. The rainfall-runoff gauging station established in 1959 has been discontinued for the winter months but will be returned to service in early spring of 1960. Some mechanical difficulties were experienced this year, so that not many rainstorms produced reliable records.

More accurate measurements of the physical characteristics of the test area will be made. The writer also hopes that some on-the-site infiltration-capacity tests may be initiated and a means found to determine the actual value of depression storage.

With regard to simplification of office design procedures, it has been the writer's hope that, with the aid of the electronic computer, now intermittently available, a large number of charts of the types shown in Fig. 17, 18 and 20 of the paper can be produced, embracing the maximum expected ranges of each of the important variables in the rainfall-runoff relationship, and all combinations thereof. It would be assumed that such simplified charts would be directly usable by the designer for areas up to at least 320 acres, or, as assumed in the paper, up to 640 acres.

With such a series of charts in hand, the designer, in selecting the proper chart would ask questions of himself such as the following:

1. What intensity-duration-frequency curve shall I use?
2. What shall I assume with regard to chronological location of peak rainfall to the total rainstorm?
3. What are the shape and numerical values of the infiltration capacity curve?
4. What is the value, in inches, of depression storage?
5. What are the factors affecting overland flow to the street gutter?
   a) The ground slope.
   b) The surface frictional coefficient.
6. What percentage of the drainage area is "directly connected impervious area?"
7. What percentage of the drainage area is "indirectly connected impervious area?"
   a) Connected via street gutter?
   b) Connected via turfed areas to street gutter and then to sewer?

Question 7 is not pertinent to the Chicago problem since all roofs, parking lots, etc. are required, by ordinance, to be directly connected to the sewer system, but the question may be pertinent in other cities. Question 7 (b) would often apply to suburban areas.

Such an extensive series of charts would not be feasible but for the advent of the electronic computer. Unfortunately for this program in Chicago, the electronic computer purchased by the Bureau of Engineering has been put in use in numerous other fields and the production of an extensive series of runoff charts has been delayed.

The charts would be arranged in groups and cross-indexed for ease of reference, so that the designer could pick whatever combination of values of the principal variables that would fit the rainfall, physical characteristics and land use of the particular drainage area. Future research would be directed at verification of the component values and the hydraulics of overland and channel flow.

For larger areas, areal distribution of rainfall must be considered and hydrographs of the smaller tributary areas synchronized to determine downstream hydrographs.

In conclusion, the writer wishes to state again that to obtain a fuller understanding and, eventually, a simplified method for evaluation of the rainfall-runoff relationship in urban areas, we must not only know what happens but why. Gaugings could answer the "what," if we had enough gaugings, but the "why" must be dealt with by the methods of microscopic analysis, followed by

a synthesis of the component parts, with the final synthesis verified by gaugings at carefully selected locations. Only then can we proceed with any high degree of certainty to design an initial system of sewers in an ungauged area. Otherwise, as we approach a new sewer problem, we are confronted with the dilemma of two opposing questions. "How can we design this sewer system without a gauging?" and "How can we gauge the runoff without a sewer system?"

The coefficient in the rational method is, on the other hand, too easily "rationalized" downward to meet available dollars, whole still purporting to give protection from a storm of a given frequency. The frequency of occurrence affects only the "I" in the rational method. The "C" is flexible and mysterious.

CLINT J. KEIFER,[2] M. ASCE.—The writer is grateful to those who have taken part in the discussion of the subject paper. Their constructive comments are well appreciated and should enhance the value of these studies.

Mr. Wolf's discussion pertains mostly to the design storm pattern which has been covered more thoroughly elaswhere.(1)

The Eltinge-Towne formula, Eq. 3, currently used in Chicago, compares closely to Yarnell's curves published in the U.S. Department of Agriculture, Misc. Pub. #204, August, 1935, and to the more recently published curves in the U.S. Weather Bureau, Technical Paper No. 25, 1955.

In latter publication, rainfall-frequency-duration curves for over 200 localities in the United States have been plotted and very few take the straight-line shape as suggested by Mr. Wolf.

Mr. Wolf states that no great numerical change would result if Eq. 33 were to be substituted for Eq. 3. This is probably true for most durations of storms including the very short durations. The high-intensity rainfall at the peak for the short duration would not greatly affect the peak runoff because routing through overland flow and gutter storage would immediately attenuate such peak.

However, for the duration of 180 min, Eq. 33 would produce an average rainfall intensity one third greater than obtained from Eq. 3. Due to the constancy of the volume of abstractions of infiltration and depression storage, the resulting runoff rates would be considerably more than one-third greater than the runoff obtained from Eq. 3.

Messrs. McPherson and Willis point out that the overland-flow relationships are based on experimental tests on sprinkled plots and are possibly a "gross" over-extension of the original experiments. It is true that the variation of cover, slope, length, etc., on city lots are unlike the homogeneous conditions used in the tests. However, the writer believes that the average conditions on hundreds of lots can be closely represented so as to produce the correct detention-runoff relationship with the equations derived in Reference 6.

As requested by Messrs. McPherson and Willis, a re-run was made of type 5 land use under Fig. 20 (a), restrictions for the case of roof drains connected to the gutters. The results were not significantly different from those shown for Type 5 land use in Fig. 17. To obtain a peak runoff rate 160% greater than the 1.8 cfs per acre shown in Fig. 17 as determined by the Johns Hopkins Method, all private walks and driveways would need to be treated as directly

[2] Engr. of Sewer Hydr., Bur. of Engrg., Chicago, Ill.

connected impervious surface and the infiltration on pervious areas would have to be greatly reduced.

Messrs. McPherson and Willis request that inlet hydrographs would be helpful in comparing methods. Shown in Fig. 31 are the inlet hydrographs at the front and side street catch basins. These inlet hydrographs are multiplied by their total tributary areas and added to the roof hydrograph to form the inflow into the lateral sewer system. After routing this lateral inflow hydrograph 7-1/3 min by time offset, the lateral outflow hydrograph shown in Fig. 18 is obtained.

In Figs. 20 and 21, the dotted portion of the curves were extrapolated by arbitrarily assuming that for completely impervious surfaces, the runoff would be 90% of the average rainfall rate. For the average rainfall rate the maximum period of rainfall is taken 10 min greater than the indicated time of travel.

Messrs. McPherson and Willis would like to have more coverage given to the part on areal distribution of rainfall. For the derivation of the curves shown in Fig. 24, Mr. Eltinge took 22 storms having volumes "arbitrarily limited to those exceeding 1.0 inch at 30 minutes, 1.25 inches at one hour, 1.50 inches at 2 hours and 1.7 inches at 3 hours." For each rainfall he took the gauge giving the maximum reading for an hour period as the storm center. He then plotted the intensities of the other sixteen gauges for the same hour period against their distances from the storm center. Similar graphs were plotted for 30 min, 2 hr, and 3 hr periods. From these graphs he derived the curves shown in Fig. 24. Mr. Eltinge then compared his results with data given by Frank A. Marston, Transactions ASCE, page 535, 1924.

The derivation of Fig. 25 from the curves shown in Fig. 24 was as follows: Each of the curves in Fig. 24, limited to a given distance from the storm center, was rotated about the vertical axis forming a volume of revolution. This volume was then divided by its base area giving the average rainfall factor for circular areas.

No allowance for areal distribution of rainfall was used in deriving the curves shown in Fig. 17, 18, 19, 20, and 21. As can be seen in the computation form Table VII, Col. 17, the factor "K" is applied to the runoff after determining the quantities of runoff based on the intensities at the storm center.

It is true that in comparing the hydrograph method with the "rational" method in Fig. 19, the values of "C" inherently include an areal distribution factor if they were derived by dividing the peak runoff gauged by the average rainfall from a number of rainfall gauges evenly distributed over the drainage area. However, if a single gauge was used, the "C's" would not necessarily reflect an areal distribution factor.

With reference to Fig. TK, Messrs. McPherson and Willis show that the authors' method produces a time-displacement of the peaks of 0.7 of the time of travel in the sewers, while Hicks used 2/3 and the Johns Hopkins data produces 0.8. The writer believes the shape of the lateral hydrographs will influence the slope of the lines in Fig. TK. In Fig. 32 two triangular-shape lateral hydrographs, one having a delayed peak, and the other having an advanced peak, are routed for 15 and 60 min time of travel, by the method set up in Eq. 31. It can be seen in Fig. 32 that for the delayed-shape lateral hydrographs, the time of travel of the peak is very short, thus indicating a high velocity flood wave. For the advance-shape lateral hydrographs, the time of travel of the peak is much longer, thus the velocity of the flood wave approaches the velocity of the water particals. By this procedure, it can be shown that the time-displacement of the peaks to the time of travel in the

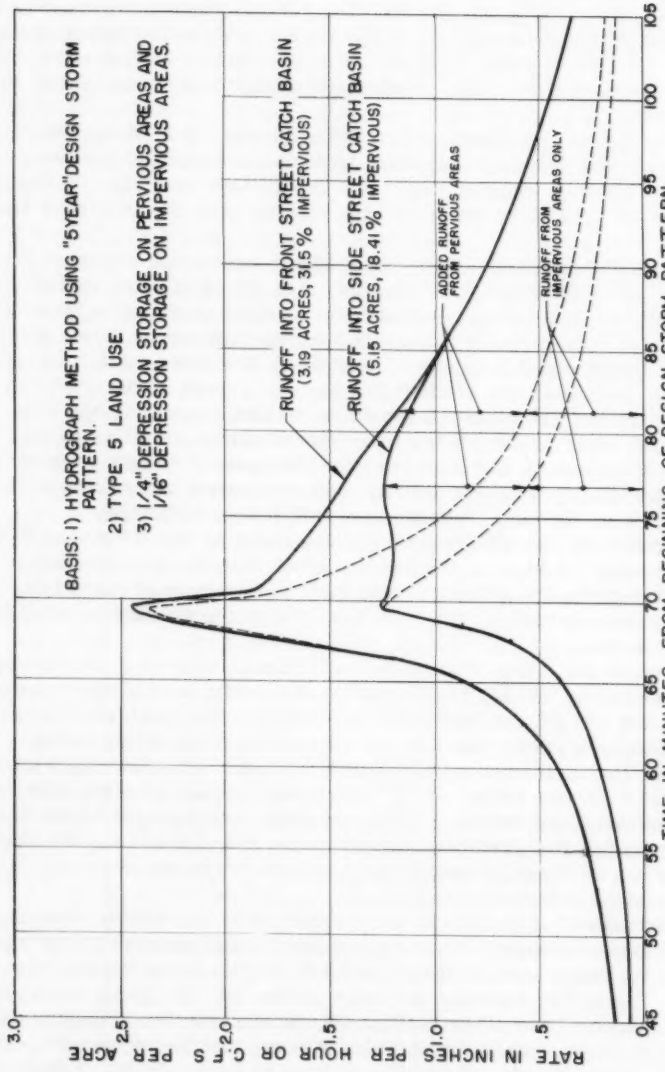


FIG. 31.—INLET HYDROGRAPHS AT FRONT AND SIDE STREET CATCH BASINS

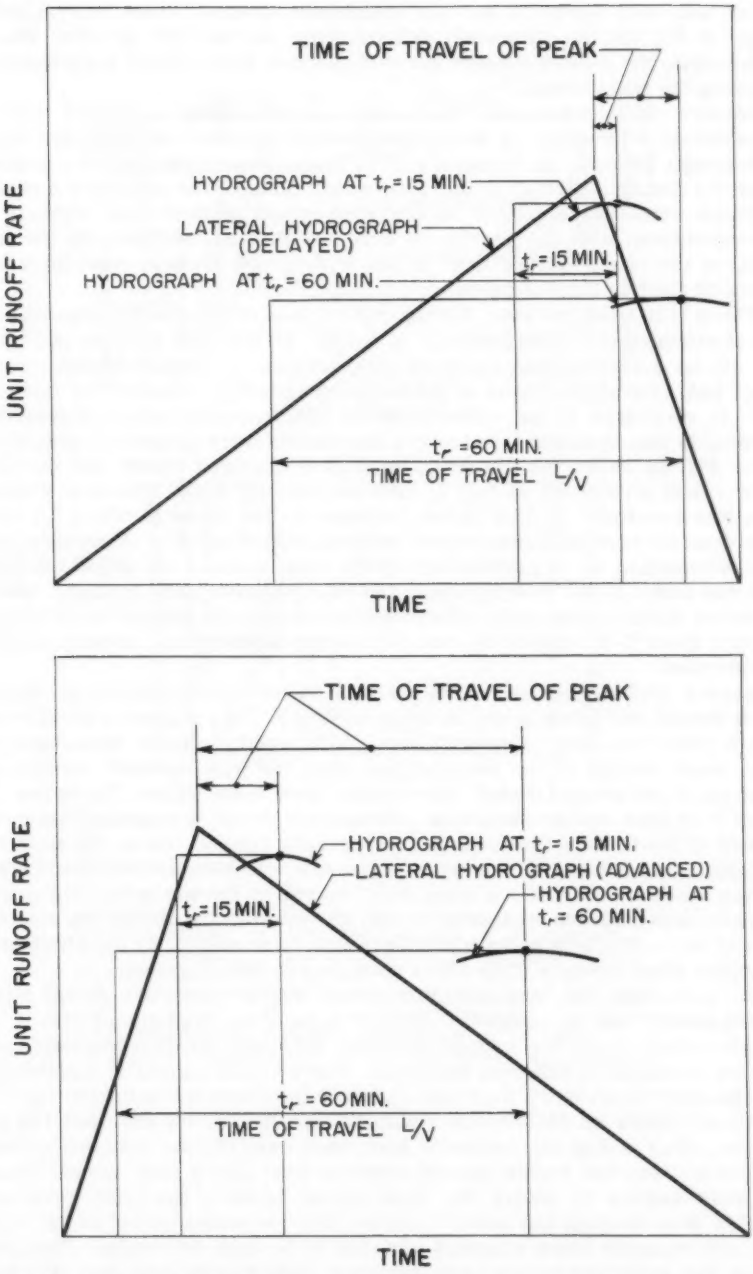

TIME OF TRAVEL OF PEAK
HYDROGRAPH AT t_r = 15 MIN.
LATERAL HYDROGRAPH (DELAYED)
HYDROGRAPH AT t_r = 60 MIN.
t_r = 15 MIN.
t_r = 60 MIN.
TIME OF TRAVEL L/V
UNIT RUNOFF RATE
TIME
TIME OF TRAVEL OF PEAK
HYDROGRAPH AT t_r = 15 MIN.
LATERAL HYDROGRAPH (ADVANCED)
HYDROGRAPH AT t_r = 60 MIN.
t_r = 15 MIN.
t_r = 60 MIN.
TIME OF TRAVEL L/V
UNIT RUNOFF RATE
TIME

FIG. 32.—SCHEMATIC DRAWING SHOWING THE EFFECT OF THE SHAPE OF LATERAL HYDROGRAPH ON TIME OF TRAVEL OF THE PEAK.

sewer will vary from 1.0 for the completely advanced shape lateral hydrograph to 0.0 for the completely delayed shape lateral hydrographs. For the latter case, the peak discharge for each section would occur simultaneously all along the main sewer.

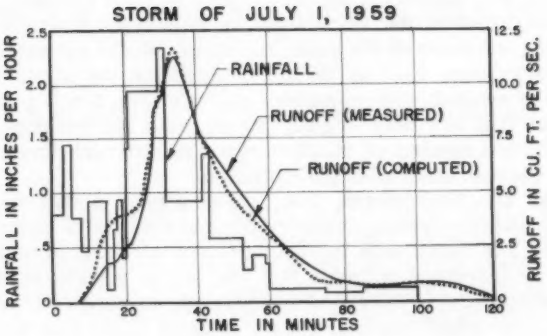
Messrs. McPherson and Willis are understandably concerned with the quantitative difficulties of the rainfall-runoff process as computed by the Hydrograph Method. In Chicago, a slow transitionary process of converting from the Rational Method to the Hydrograph Method was used over a number of years. Results calculated by the Hydrograph Method were checked for reasonableness with the previously accepted Rational Method. On the other hand, of the many factors used in the Hydrograph Method, most have been tested by field or laboratory experiments.

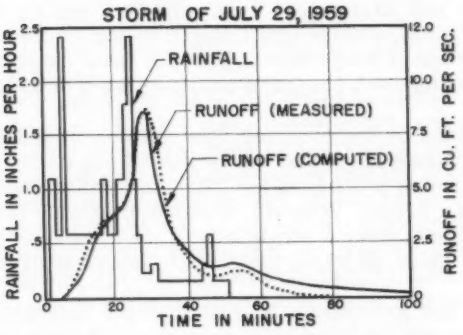
There is a need, however, for the comparison of the runoff computed from actual rainfalls with field gaugings of runoff. To this end, the City of Chicago has set up a rainfall and runoff gauging station. A tipping-bucket rainfall gauge and a parabolic flume in the field are remotely connected by telephone lines to recorders in the writer's office. Unfortunately, some malfunctions resulted in this equipment, and only a few storms were accurately gauged last year. Fig. 33 shows the gauged rainfall, the measured runoff, and the computed runoff for storms of July 1, 1959 and July 29, 1959. The total drainage area was measured at 13.9 acres tributary to the flume of which 5.7 acres were directly connected impervious surface, and 1.2 acres of impervious surface discharging on to pervious area. To make a closer fit, 50% of the latter area was added to the directly connected impervious surface making a total of 6.3 acres directly connected. Also, routing through the lateral sewer was increased from 7-1/3 min to 8 min. With these adjustments, a fairly close fit was obtained.

Messrs. McPherson and Willis question whether quantitatively the Hydrograph Method will come in use in the near future. They suggest a simpler approach based on actual measurements of the rainfall-runoff relationship as being more fruitful in the near future. Over the past hundred years, many gaugings of the rainfall-runoff relationship have been taken. The writer believes it is time now to develop in microscopic detail, a mathematical model of each of the factors involved. It is especially timely, due to the advent of electronic computers. If research engineers will take up the challenge to analyze in detail each of the many factors, and by the use of the computers, compare computed and measured runoff, the quantitative model can soon become of age. Simplifying the method can then be developed by the elimination of factors which produce little affect upon the runoff hydrographs.

Mr. Bock asks the "engineering reasons" for selecting the design storm pattern since it has no assigned frequency in the usual statistical sense.

The authors began the hydrograph study by taking uniform intensity rainfalls for a number of different durations. These "block rainfalls" had the uniform intensity equal to the 5-yr rate-duration-frequency curve for the selected durations. Each of the "block rainfalls" for 15, 30, 60, 120, and 180 min duration, after adding the estimated antecedent rainfall, was reduced by rainfall abstractions and routed through overland flow, gutter flow, lateral sewer, and main sewers to obtain the peak runoff rates of the flood wave as it traveled down through the sewer system. The "envelope curve" of the peaks from the separate block rainfalls was then to be used for design. This procedure had to be performed over again for each type of land use. The tremendous amount of labor required to perform the foregoing procedure, led the





BASIS: TOTAL AREA DRAINED ..... 13.9 ACRES
DIRECTLY CONNECTED IMPERVIOUS AREA..... 5.7 "
IMPERVIOUS SURFACES DISCHARGING INTO PERVIOUS AREAS..... 1.2 "
RAINFALL GAUGED 1/2 BLOCK FROM CENTER OF DRAINAGE AREA
RUNOFF MEASURED AT LATERAL OUTLET BY PARABOLIC FLUME
STANDARD INFILTRATION CURVE
1/4" DEPRESSION STORAGE ON PERVIOUS AREAS
1/16" " " " IMPERVIOUS AREAS
ROOF HYDROGRAPHS DIRECTLY CONNECTED TO LATERAL
TIME-OFFSET ROUTING THROUGH LATERAL SEWER OF 8 MINUTES.

FIG. 33.—COMPARISON OF MEASURED AND COMPUTED RUNOFF

writer to probe for a better method. If a single storm pattern - perhaps not occurring in nature - could be found, that, after being reduced for abstractions and routed, would produce peak runoff rates equal to the envelope curve, described above, that storm pattern could then be used for design of a sewer system and would reduce the calculation by 80%. This single storm pattern and the resulting envelope of the peak discharges would not have an assigned frequency of recurrence such as the block rainfalls, but would result in the same design. The synthetic storm pattern shown in Fig. 3 will produce a flood wave with peaks that are only slightly higher than the envelope of the peak flows derived from a number of block rainfalls of different durations, each of which have the correct antecedent precipitation.

It is true, as Mr. Bock states, that "the scatter of actual rainfalls is so wide that no meaningful typical chronological pattern can be detected." However, the engineer is faced with the problem that he must have a storm pattern. The writer therefore believes that the storm pattern should have the major factors which affects the runoff hydrograph, namely: the average rainfall, the correct antecedent rainfall, and the location of the peak as determined from statistics for each duration.

Messrs. Izzard and Armentrout have combined routing through overland flow detention and gutter storage into a single routing procedure. This greatly simplifies that portion of the problem. Also, by their method, storage curves need not be redrawn for each change in constants, such as length, slope, roughness, etc.

In Eq. 25b, storage on the overland flow surface plus that in the gutter is assumed to be a function of rainfall minus abstrations (I) and the discharge at the inlet (Q). The routing procedure, Eq. 27b, can be used where the flow from one surface enters a gutter. However, when the flow from two surfaces, such as pavement and lawns, enter a single gutter, the net supply cannot be represented by a single value (I) since the rainfall minus abstrations is different for each surface.

Messrs. Izzard and Armentrout have applied the hydrograph method using their simplified routing procedure to compare computed runoff with inlet gaugings at Baltimore. This study resulted in a fairly close agreement between computed and measured runoff.

In making these studies Messrs. Izzard and Armentrout came to the conclusion that "the effect of abstrations from rainfall is important and is dependent in large measure on antecedent rainfall." The authors have had the same findings in their studies using the hydrograph method. If these findings are correct that abstractions and antecedent rainfall play such an important part in the rate of runoff, should not an attempt be made to study these factors in greater detail?

OPERATION OF A 7-MILE DIGESTED SLUDGE OUTFALL[a]

Discussion by C. H. Lawrance and D. R. Miller

CHARLES H. LAWRANCE,[1] M. ASCE, and DAVID R. MILLER,[2] M. ASCE.—This paper has been most interesting to the writers, who were among the designers of the facility discussed. It is particularly gratifying to note the significant improvement observed in the incidence of floatable solids of sewage origin at beach sampling stations and the general improvement in the conditions of the receiving waters after the outfall was put into operation. Because of the extensive monitoring program of the City covering both shore and offshore stations in Santa Monica Bay, additional valuable data and conclusions may be expected in years to come concerning submarine disposal of sludge and of treatment plant effluent.

The decision to replace the filter and drier facilities at Hyperion with ocean disposal was one of the most controversial aspects of the entire Hyperion Expansion Program. Many persons argued that it was an inconceivable waste of a valuable natural resource; others that the discharge of this material into a Bay having no strong, dominant offshore currents would result in an intolerable pollution of the Bay. Still others contended that a separate sludge facility was a sheer waste of money and that the sludge should be discharged with the effluent through the Ocean Outfall for Effluent Disposal. The authors have presented the arguments refuting these contentions. The writers desire to elaborate upon some of the points at this time.

The fertilizer project to which the authors alluded was given every chance to succeed, for a number of years, in the face of obstacles that arose in solids-removal and in processing and marketing the dried sludge. Since sewage sludge is an unavoidable by-product of the larger overall process of sewage treatment, the cost of its disposal is rightfully apportioned within the total cost of sewage treatment. If it is possible to utilize it directly as fertilizer on land - as opposed to fertilizer for marine life - and if revenues are obtainable to defray the costs of its disposal, so much the better. However, sewage sludges, by their very nature, are not in a position to compete openly with inorganic commercial fertilizers on the basis of fertilization potential.

Hyperion fertilizer contains approximately 2.5% nitrogen, 3.5% phosphoric acid, and no potash. Digestion operations affect the nitrogen content, as described by W. F. Garber, M. ASCE.[3] A typical commercial inorganic fertilizer, 8-8-4, sells in bulk in the Los Angeles area for about $47 per ton,

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[a] July, 1959, by N. B. Hume, R. D. Bargman, C. G. Gunnerson and C. E. Imel.

[1] Proj. Engr., Koebig and Koebig, Cons. Engrs., Los Angeles, Calif., formerly San. Engr., Hyperion Engrs., Los Angeles, Calif.

[2] Proj. Mgr., Daniel, Mann, Johnson and Mendenhall, Archt. and Engrs., Los Angeles, Calif., formerly Proj. Engr., Hyperion Engrs., Los Angeles, Calif.

[3] "Plant-Scale Studies of Thermophilic Digestion at Los Angeles," by W. F. Garber, Sewage and Industrial Wastes Vol. 26, No. 10, October, 1954, p. 1202.

representing a price of $588 per ton each of nitrogen and of phosphoric acid, Another commonly used fertilizer, 16-20-0, sells for about $80 per ton, corresponding to $500 per ton of nitrogen and $400 per ton of phosphoric acid. Based upon these fertilizer indices, a comparable sales price to the bulk consumer would be about $15 per ton for the Hyperion dried sludge. The disparity between this figure and the $4 per ton for which the sludge is actually sold to a fertilizer firm for subsequent marketing apparently represents handling, transportation, and other charges on the part of this firm and reflects some of the difficulties in marketing the dried sludge. Potential markets have included crop-growers within the Los Angeles Basin and environs, including even those located in the Imperial Valley some 200 miles southeast of Los Angeles. However, for a variety of reasons ultimately determined by the economics confronting the grower applying the fertilizer, inorganic fertilizers have generally been found preferable despite the soil-conditioning advantages of the dried sludge, and this market has never fulfilled expectations. Potential retail markets for sale of sewage sludges to householders have encountered sales resistance. Problems in the production of the sludge, particularly in the elutriation, conditioning, and vacuum filtration processes at Hyperion have been presented comprehensively in the literature by R. D. Bargman, F. ASCE, W. F. Garber, M. ASCE, and J. Nagano.[4]

The recent experiences of the City of San Diego, Calif. as described by R. C. Merz, F. ASCE[5] have shown that it is more economical for San Diego to dispose of digested sludge from its primary treatment plant onto waste land as a reclamation project for agricultural purposes than to operate its existing sludge conditioning and drying facilities. The processed sludge was formerly being put to directly beneficial use and the City's decision to continue such use, in the modified manner selected, represents an encouraging reversal of the trend towards disposal of the sludge by unreclaimable means and is a demonstration of the fact that under the right circumstances, this natural resource can be exploited profitably. In the case of San Diego, the City already owned available waste lands and was able to transport the liquid digested sludge to them by tank truck for subsequent spreading.

Reportedly the cost of fertilizer production had been $39.15 per ton of dry solids handled in operations similar to those for which the authors have cited their costs of $14.22 at Hyperion. At San Diego, the changeover to land disposal from the original method of fertilizer production for contract sale to a fertilizer firm has resulted in a savings of $16.12 per ton of dry solids, leaving a cost of $23.03 per ton for land disposal. The San Diego plant handles approximately 60 mgd as compared to Hyperion's 270 mgd; this flow difference is undoubtedly a major factor entering into the differential in unit treatment cost between the two plants.

The overall environmental picture, embracing the aspects of water, air, and food supply is gradually being drawn more and more into focus in the eyes of the engineer as populations swell and by their growth point to a possible eventual saturation point in certain areas. Mark Hollis[6] among others has sounded a warning regarding the outlook for water pollution in the United States. Others may be expected to follow. The author's own observation that man

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[4] "Sludge Filtration and Use of Synthetic Organic Coagulants at Hyperion," by R. D. Bargman, W. F. Garber, and J. Nagano, Sewage and Industrial Wastes Vol. 30, No. 9, September, 1958, p. 1079.

[5] "Utilization of Liquid Sludge," by R. C. Merz, Water and Sewage Works, Vol. 106, No. 11, November, 1959, pp. 489-93.

[6] Engineering News-Record, Vol. 163, No. 17, October 22, 1959, p. 24.

sometimes exchanges one pollution problem for another is poignantly true in the case of the Los Angeles Basin. In this area not only has the Hyperion Treatment Plant been squeezed between air pollution limitations from sludge-drier stack emissions on one hand the water pollution limitations from sludge elutriate discharge on the other hand, but refuse incinerators for individual dwellings are now extinct and municipal incinerators are gradually giving way to land disposal which may in itself present some potential problems of ground water pollution.

It is difficult to compare the operating costs of the two methods of sludge disposal, fertilizer production and submarine discharge, because of the many factors involved. Some of these are tangible, such as power consumption and availability, and some tend to be intangible, such as effects upon other plant processes and effects upon the receiving waters. A crude economic comparison for the best year in solids removal with 92 tons per day or 57.5% of the total digested sludge production being processed and the remainder being lost to sea via the one-mile ocean outfall would show a cost of the processing at $14.22 per ton equal to $1,318 per day. This would compare with 160 tons per day at $1.15 per ton or $185 per day for sludge outfall discharge. A strong advantage in favor of the sludge outfall is shown. Addition of the fixed charges to the comparison would probably add more to the advantage of the sludge outfall.

The cost of ocean disposal will ultimately be increased because of the cost of equipment which will be required to remove floatables, but this will not materially change the sludge outfall's economic advantage. These floatables, originating with the sludge outfall discharge and recognizable only by trained personnel as being of sewage origin, have already been reduced in their incidence at beach sampling from 1.04% to 0.07%. In the face of these excellent operational results, the fact that the City of Los Angeles still plans to go ahead and spend an additional $200,000 to remove even this small amount of residual floatables is a testimony to the desire of the City to do its utmost to maintain the best possible conditions in the waters of Santa Monica Bay.

Present pilot plant work on removal of floatables from digested sludge employs a compartmented rectangular tank in which the digested primary sludge is diluted with plant effluent and then is successively subjected to rapid mixing, slow stirring, and sedimentation to promote separation of the fractions which appear as floating material on the ocean's surface from time to time. The most conspicuous material is rubber in the form of thin flecks and occasionally fragments of rings originating with rubber goods. Cellophane particles are quite commonplace, but these exhibit a tendency to settle as well as to float both in sea water and in fresh water. Large volumes of hair occur in the sludge along with other constituents such as humus, grease particles, undigested vegetable matter, and occasional cigarette filter tips. Following rapid and slow mixing in the tank, the worst offender, rubber goods, is largely floated off as effluent scum in the sedimentation compartments along with considerable hair and grease. The sludge humus mostly settles to the bottom, but the cellophane, hair and vegetable matter may be found in both the tank scum and bottom sludge and even, to a lesser extent, in the intermediate depths of the tank. Work is now proceeding to establish better the relationships between removal of floatables and such parameters as power application and overflow rate, prior to development of final design.

The design criteria for the sludge outfall were established only after exhaustive studies had been made upon the settling characteristics of the digested sludge and the oceanographic conditions of Santa Monica Bay. These

latter conditions, as summarized by the authors, are typified by onshore counter-clockwise surface currents and currents at depth which are generally parallel to shore and are relatively slow. The existence of convergences offshore is noted with the observation that these seem to act as barriers to prevent floating material from reaching the beaches. While occasional swimming and other aquatic sports activity extend for an appreciable distance offshore, by far the majority of use occurs normally within 1,000 ft of shore. The convergences have a beneficial effect in discouraging floating material from entering this nearshore zone.

The oceanographic studies measuring temperature variations in the area of proposed sludge discharge and determining the subsurface current pattern were made for the purpose of establishing the required length of the outfall and the depth of discharge necessary. The intention was always to discharge the sludge at sufficient depth to provide mixing with cold bottom water so that the resulting density of the sludge-sea water mixture would be greater than the density of the overlying warmer water and the sludge field would be kept completely below the surface. This prevents any possible unsightliness at the surface and keeps the mixture away from the stronger surface currents. Figs. 6 and 7 illustrate how this has occurred under actual operation and how the sewage field is generally kept below 150 ft in depth. These figures illustrate conditions under typical summer temperature variations. It would be interesting to see what happens in the winter season when the temperature at the point of discharge is 5°F. cooler than the surface water.

Figs. 6 and 7 are also interesting from the standpoint of suggesting the dispersion of the pollutants upon submarine injection. The temperature contours of Fig. 7 shows the local heating effect of the sludge outfall discharge as evidenced by the general downward depression of the contours in the vicinity of the outlet and coliform field. Immediately above the outlet there is an upward bulge of temperature contours, apparently reflecting an induced upward movement of the cold bottom water next to the rising jet of the discharge. Within this bulge, the "inversion" or isothermal lens at 54°F. would seem to represent the rising mixture of outfall discharge and surrounding sea water. The sampling traverse was taken at an angle of 45° from the direction of the sludge discharge, following the direction of the subsurface current. The rising jet may not have been fully intersected by the profile at all points, but the profiles drawn appear quite representative.

The complete submergence of the coliform field deserves some comment. The physical dilution of the submerged "sludge field," particularly that area directly over the outlet, is very likely considerably less than that suggested by the coliform density contours in relation to the coliform density of the injected flow. This sludge and diluent mix prior to discharge probably had a coliform concentration of slightly above 100,000 coliforms per ml, based upon the usual values experienced at the Hyperion plant. Therefore, the "apparent" dilution is up to about 100,000 at the upper limit of the coliform field, 10,000 at the 10 coliforms per ml contour, 1,000 at the 100 per ml contour, and somewhat less than 1,000 at the bottom of the isothermal lens. The actual physical dilution could be computed from salinity and temperature data and can be approximated by methods employed by N. H. Brooks, A. M., ASCE[7] and by the writers.[8] Tentatively, the physical dilutions at the bottom of the isothermal

[7] "Depth of Discharge from Hyperion Sludge Outfall," by N. H. Brooks, Memorandum Report, November 26, 1955.

[8] "Ocean Outfall Design," Hyperion Engrs., 828 S. Figueroa St., Los Angeles, Calif., October, 1957.

lens, the 100 per ml contour, the 10 per ml contour, and the 1 per ml contour would appear to be in the order of 30, 75, 100, and 150, respectively, these values representing the rising column issuing from the outlet. At a distance horizontally away from the column the physical dilution would be appreciably greater than in the column because of lateral diffusion experienced once the column has been sufficiently slowed in its ascent by acquiring a density approaching that of the surrounding sea water. This has apparently happened at about 250-ft depth, the approximate location of the isothermal lens and the downstream "bulge" of the coliform contours. The narrower the initial "field," the more rapid will dilution take place by the action of turbulent diffusion. The single outlet discharge into the cold bottom water would seem to reach a density equilibrium with its surroundings before it established a very wide column base or field width, hence its lateral diffusion should be reasonably rapid.

Upon establishment of the field, the complex factors of sedimentation, physical dilution, and mortality come into play in the reduction of coliform densities. These have been discussed previously by C. G. Gunnerson, M. ASCE[9] and by the writers.[10] In the case of Fig. 6, the writers have insufficient information to offer anything more than generalizations regarding the nature of the coliform contours. Downstream of the outlet, the distance between contours of one magnitude (90% reduction) is about 3,000 ft, especially at the "bulge." This is believed to correspond to 3 to 5 hr time in the direction of the current. Assuming a stratification of field initially at about 230-ft depth, equivalent to a stratified sewage field on the surface of the ocean, the overall T-90 or time for 90% coliform reduction would be about 4 hr. Sedimentation of coliforms takes place rather slowly judging by the spread of the contours. This would imply that the sludge suspension itself settles fairly slowly also. Additional bottom core samples and trawls should clarify this matter. The writers are unable to explain the seeming discrepancy between expected physical dilution and the "apparent" bacterially-measured dilution of the outfall discharge in the depths lying more or less directly over the outlet. It may be that the subsurface field stratifies fairly rapidly except for a small portion of rising column which carries up to above the 200-ft depth. In any event, local coliform concentrations much higher than 100 per ml over the outfall would be expected unless the sampling chanced to miss them.

The authors state that after one year's operation, the Darcy f of the line has increased 23%. Even with the increase, the line still has adequate capacity, and the decision to clean the line mechanically was apparently a routine maintenance procedure. Since this cleaning has now taken place, it would be interesting to know how much the cleaning was able to reduce the friction value and if there is any tendency for an immediate return of the deposition of grease which the authors assume is the cause of the increased roughness. Is there any plan for the regular use of the cleaning device on a regular maintenance program?

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9 "Sewage Disposal in Santa Monica Bay," by C. G. Gunnerson, Transactions ASCE, Vol. 124, 1959, p. 823.

10 Discussion, by C. H. Lawrance and D. R. Miller, Hyperion Engrs., 828 S. Figueroa St., Los Angeles, Calif., October, 1957.

WASTE WATER RECLAMATION FOR GOLF COURSE IRRIGATION[a]

Discussion by Ralph Stone

RALPH STONE,[1] F. ASCE.—Existing engineering experience with wastewater reclamation for golf-course irrigation has been masterfully summarized by Mr. Merz. The writer has had the privilege of working on golf-course irrigation systems using reclaimed water both in conjunction with Mr. Merz at the University of Southern California and as a design consultant.

A common place difficulty in the use of reclaimed water is to consider effluent to have the same quality and character as drinking water. This is normally because the health department checks the bacterial water quality to determine if a health hazard exists and if it is approved, no further consideration is given. The public-health concern may lead to excessive chlorination which causes corrosion of piping and interferes with grass growth. It is therefore important to utilize long-period storage and chloramine contact time to achieve satisfactory bacterial quality.

The mineral content of a reclaimed water will normally provide considerable sodium, potassium, nitrogen, phosphate, and other ions. The fertilizer effects can be used as a substitute for commercial fertilizer. The use of routine commercial fertilization plus reclaimed water may permit "burning" of vegetation. The soil colloids will control applicable irrigation rates, soil percolation, and plant growth. Reclaimed water of a high sodium content is difficult to use on a clay soil. Germination of grasses and shrubs on impervious soils with reclaimed water may present a problem. Drainage devices including importation of sandy soils may be resorted to in order to achieve soil permeability.

Certain types of industrial wastes such as boron or chlorides should be eliminated from the sewage system. Leaching may be needed to remove the existing saline content in the golf course soils. A suitable irrigation schedule for leaching irrigation waters and avoiding salt buildup is desirable for good turf culture. Over-irrigation is equally unsatisfactory since various plant diseases, including rotting, may result. Obviously then, scheduled irrigation programs are a prerequisite for optimum grass growth.

The amount of organic content present in a reclaimed water and its biochemical stability would effect the aesthetic success of using this irrigation water source. Most biological sewage treatment processes do not eliminate detergents. Foaming problems are a serious nuisance to be avoided by proper engineering designs, including suitable treatment methods, submerged piping inlets and other techniques. Algae and bacterial reservoir growths are another source of organics that must be eliminated from the irrigation supply.

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[a] November, 1959, by Robert C. Merz.

[1] Cons. Engr., Ralph Stone and Company, Beverly Hills, Calif.

The irrigation system itself should be so designed that there are no dead-ends, and deposition of organics is avoided. Corrosion problems should also be controlled in the irrigation water distribution system.

Automatic devices for irrigating are more important with reclaimed water than with conventional water sources. Irrigation with reclaimed water in the early hours of the morning can be readily achieved with an automatic sprinkler system. It is aesthetically desirable to minimize daytime reclaimed-water irrigation.

A basic need in planning the reclaimed-water system is to provide for adequate storage, bypass, and effluent disposal of these waters during rainy periods.

Unfortunately many golf courses are designed without proper engineered planning for waste-water reclamation and the application is not particularly successful.

Experience indicates that successful irrigation with reclaimed water requires specialized control of planning, construction, and operation. It is necessary to integrate the aesthetic requirements of a golf course while: (1) maintaining full protection of the public health, (2) eliminating sanitary nuisances such as odors, turbidity, and color from the water, (3) controlling deleterious minerals, and (4) providing this reclaimed water at a cost lower than that for competing fresh waters. Low cost waste-water reclamation is practical and has been achieved in varying degrees at many locations. It is expected that the successful progress in reclaiming waste water for golf course irrigation will result in greater public acceptance of this practice.

Mr. Merz is to be commended for his many contributions to waste-water reclamation studies, summarized, in part, by his discussion of waste-water irrigation on golf courses.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

VOLUME 85 (1959)

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998(SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007(HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

JUNE: 2048(CP1), 2049(CP1), 2050(CP1), 2051(CP1), 2052(CP1), 2053(CP1), 2054(CP1), 2055(CP1), 2056(HY6), 2057(HY6), 2058(HY6), 2059(IR2), 2060(IR2), 2061(PO3), 2062(SM3), 2063(SM3), 2064(SM3), 2065(ST6), 2066(WW2), 2067(WW2), 2068(WW2), 2069(WW2), 2070(WW2), 2071(WW2), 2072(CP1)[c], 2073(IR2)[c], 2074(PO3)[c], 2075(ST6)[c], 2076(HY6)[c], 2077(SM3)[c], 2078(WW2)[c].

JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087(SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096(EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134(SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184(WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330(CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

c. Discussion of several papers, grouped by divisions.

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PART 2

MARCH 1960 — 8 NO. SA 2
VOLUME 86 PART 2

Your attention is invited

NEWS OF THE SANITARY ENGINEERING DIVISION OF ASCE

JOURNAL OF THE SANITARY ENGINEERING DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

March, 1960

NEW VISTAS IN SANITARY ENGINEERING

Traditional sanitary engineering practice has been concerned with water supply and waste disposal. In recent years air pollution and radiological health have come to the forefront in sanitary engineering practice, and with this rise has come new terms, new problems, and new solutions in efforts to mitigate the effects of man's use of man's environment.

Largely unmet and often studiously ignored have been the problems of controlling natural occurrences and factors that affect man's environment. The article below on a project to conserve and improve the quality of water in the Arkansas and Red River basins describes preliminary work to control pollution largely of a natural origin.

ARKANSAS-RED RIVER BASINS WATER QUALITY CONSERVATION PROJECT

This project was undertaken by the Public Health Service in June 1957 (under authority of the Federal Water Pollution Control Act) to carry out recommendations of the Arkansas-White-Red River Basins Interagency Committee Report of January 1957 (Senate Document No. 13, first session, 85th Congress). The purpose and objective of the project is to determine the factors which cause degradation of the water in the Arkansas River and its tributaries between Little Rock, Arkansas, and Great Bend, Kansas, and the Red River and its tributaries above Shreveport, Louisiana, and to make recommendations for remedial measures. The project has been programmed in four phases.

Phase 1 - This phase involved the accumulation of all known information on water quality, including certain analyses and determinations of gaps in the existing data.

Phase 2 - This phase involves the collection of field information necessary to fill in the gaps found during Phase 1.

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Phase 3 - This phase involves the development of the recommendations for corrective action, together with economic data necessary to indicate the cost of such remedial measures as are considered necessary.

Phase 4 - This is the post investigational phase dedicated to the implementation of recommendations developed by the Public Health Service. Construction of engineering structures necessary to remove or control the causes of pollution will be the primary responsibility of private and public agencies other than the Public Health Service.

Discussion and Findings

The project is headquartered in the PHS Regional Office, Region VII, Dallas, Texas. Field offices have been established at the North Campus, University of Oklahoma, Norman, Oklahoma, and in Wichita Falls, Texas. These three offices, with a combined staff strength of 14 engineers, are now engaged in detailed field investigations of finding and measuring sources of pollution and collating essential water quality data. Agreements have been made with the U.S. Geological Survey, the Kansas State Department of Health, the University of Oklahoma, and North Texas State Teachers College to obtain (1) information relating to the geology of the area from which the natural pollution stems, (2) information on problems of reinjection of oil well brines, (3) continuous water discharge and quality data, and (4) quality data in Lake Texoma.

The findings to date indicate that the heavy mineralization of the streams, principally sodium chloride, is the major cause of the poor quality water in the Arkansas and Red River Basins. The findings further indicate that the sources of this mineralization are primarily two. One source originates in natural salt springs located on approximately a direct line from Paducah, Texas, approximately 120 miles west of Wichita Falls, Texas, to Great Bend, Kansas, located approximately 100 miles northwest of Wichita, Kansas. The second source of mineralized matter is from oil brines pumped to the surface with oil. These latter brines occur across the eastern half of Oklahoma, southeastern Kansas, and northern Texas.

It has been determined that most of the natural salt is coming from relatively small quantities of water reaching the surface through springs or seeps, located on most of the upper tributaries of the Red and central tributaries of the Arkansas Rivers. The flow from these springs and seeps is very small, ranging from a mere trickle to approximately four cubic feet per second flow. The salt content of these flows varies from a less than sea water to a saturated brine solution. The total volume of water issuing from these sources is quite small. This fact immediately suggests that it would be both practical and economically feasible to isolate these areas in some manner, thus preventing this highly concentrated flow of salt from entering the fresh water streams.

In cooperation with State pollution control authorities, considerable progress has been made in finding and measuring flows of salt brine from oil production areas. The areas where large contributions are being made have been delineated by gross procedures which are now being refined to pinpoint the specific sources of the pollution.

Analytical studies are also in progress relating the hydrology of the basins to the quality for the purpose of determining control measures including regulated stream flows necessary for obtaining the best water quality possible at points of use.

Sources of about two-thirds of the salt load carried by the two river systems have now been found. The findings indicate that it may be possible to begin experimentation with corrective natural brine pollution. Since the Public Health Service does not contemplate constructing the corrective measures, the Federal, State or local construction agencies will build the necessary engineering structures.

The project is proceeding as programmed and funds expended to date have been $83,000 in FY 1958; $63,000 in FY 1959; and $400,000 in the current FY 1960. Estimated needs for FY 1961 are $400,000 and for FY 1962 $100,000.

Future plans include completing the Arkansas-Red River Water Quality Conservation Project on schedule, and the early encouragement of the construction of corrective measures as fast as funds will permit the construction agencies at local, State, and Federal levels to participate. Plans also contemplate that procedures designed to remove oil well brine pollution will be promulgated and action instigated to reduce the polluting elements from these sources in cooperation with the State water pollution control agencies.

Improvements in the water quality of surface waters in the five States covered by the study will enable the area to expand industrially, agriculturally, and in population. The benefits from the total water resource development already underway in this area will increase materially. The flow of water affected by salt degradation is estimated at over 30 million acre feet per year.

Did You Know That --

Dr. Werner N. Grune was recently promoted to the rank of full professor of Civil Engineering at the Georgia Institute of Technology, Atlanta, Georgia. Dr. Grune who came to the Georgia Institute of Technology in the fall of 1952, has devoted most of his time to sanitary engineering teaching and research on sewage and industrial waste, especially atomic energy waste disposal on which he has worked since 1948.

The Board of Control of the Water Pollution Control Federation approved South Dakota as a Member Association at the annual meeting at Dallas, Texas last October. Formerly, North and South Dakota made up the Dakota Section.

The Gulf Intracoastal Waterway provides a continuous protected channel from Carrabelle, Fla. to the Mexican border, almost 1,200 miles. The channel is at least 12 ft. deep and 125 ft. wide. Of 36 U.S. ports handling three million or more tons of cargo annually, 10 are along the waterway. The Gulf waterway and the Mississippi-Ohio-Illinois River System provides an extensive system of inland waterways.

The board of governors of the National Conference on Industrial Hydraulics has appointed a new secretary, LeRoy A. Wickstrom of Chicago, an instructor in industrial engineering at Illinois Institute of Technology and associate analyst in engineering economics research at Armour Research Foundation. In addition to his new appointment as NCIH secretary, Wickstrom is also conference director for "New Ideas 1960," an industrial management engineering conference co-sponsored by Armour Research Foundation and Illinois Tech.

David S. Jenkins, formerly Director of the Office of Saline Water, Interior Department, has opened a private consulting office specializing in saline water conversion, water supply, resource surveys, and evaluations.

Dr. Haldon A. Leedy, director of Armour Research Foundation of Illinois Institute of Technology, was reelected president of the Midwest Air Pollution Prevention Association, Inc., at the group's annual meeting Dec. 16, 1959. Other officers reelected were Lewis A. Evans, Chicago & Indiana Railroad Co., first vice-president ; Dan A. Sullivan, consultant at Armour Research Foundation, second vice-president; Douglas R. Fuller, Northern Trust Co., treasurer; and Dr. Christopher E. Barthel, Jr., assistant director of ARF, secretary. Elected to the board of directors for three-year terms were Raymond T. Host, Joseph Goden Incinerators; Raymond D. Maxson, Commonwealth Edison; Erwin E. Schulze, of the law firm of Stevenson, Conaghan, Hackbert, Rooks and Pitts; Herbert A. Wolff, R. R. Donnelly and Sons Co.; and Henry C. Woods, Sahara Coal Co.

ASEIB BOARD MEETS

At the October 1959 meeting of the Board of the American Sanitary Engineering Intersociety Board, Thomas R. Camp was elected Chairman, Blucher A. Poole, Vice-Chairman, and R. S. Rankin, Secretary-Treasurer.

The Board also voted to submit an application to the National Science Foundation for support of a study conference in June 1960 of sanitary engineers and scientists.

Editors Note:

It is understood that examinations for certification by the ASEIB will be held sometime this year, but firm dates and places have not yet been selected. In view of our inability to have this information for interested readers of this Newsletter, it is suggested that those persons who wish to take the examinations this year signify their intentions in writing to the Secretary, ASEIB, 16th Floor, 33 W. 39th St., New York 18, N.Y., so that they will receive prompt notification.

NEW DIPLOMATES OF AMERICAN ACADEMY OF SANITARY ENGINEERS

The American Sanitary Engineering Intersociety Board, Inc. recently announced the names of those who have been certified as Sanitary Engineers during 1959.

Robert D. Bargman
Edward D. Bastian
Maldon B. Crabill
Anson B. DeWolf
Roy Dragone
Stacy W. Edwards
Louis J. Fontenelli
Leonard Martin Fraivillig
John Galandak
Harry H. Hendon
H. Harvey Hunt
A. Joel Kaplovsky
George J. Kupchik
Donald G. Larkin
Charles H. Lawrance
Scott Ellsworth Linsley
B. A. McAdams
Samuel C. McLendon
William A. Moggio
James B. Neighbor
Herbert H. Rogers
Alfredo Salazar
Milton Spiegel
Ralph Stone
Edward D. Thoits
Norman E. Waggoner
Raymond Walsh
Myron A. Wayne
Leonard S. Wegman
Theodore C. Williams
Seymour Zubkoff

These men become Diplomates of the American Academy of Sanitary Engineers and will be included in the 1960 Roster to be published soon.

Analysis of SED Publications

The Committee on Publications of the Sanitary Engineering Division recently analyzed the material published in the Journal during the past four years, as well as the number of sanitary engineering papers published in Transactions during the last ten years.

Several significant items are demonstrated.

1. The number of non-ASCE member authors comprises 28 percent of the total, and is exceeded only by authors in the Fellow grade.
2. While "air pollution" articles comprised 14 percent of the total published, 13 of 17 individual authors were not ASCE members.
3. Sanitary engineering articles published in Transactions over the 1949-58 period were 9 percent of the total—substantially greater than would be expected if each Technical Division were equally represented.

The two tables follow. The portion of the first table concerning author grade will not check with the total of papers due to multiple authorship.

SUMMARY OF PAPERS PUBLISHED
NUMBER AND ASCE AUTHOR GRADE
SANITARY ENGINEERING DIVISION JOURNAL
1956-1959

Article	Author Grade (Present Classification)					Publication Year				Four Year Total
Classification	Non ASCE	AM	M	F	Committee	1956	1957	1958	1959	
Air Pollution	13	1	2	1	3	3	4	5	2	14
Education	3	1	6	6	2	1	8	1	2	12
Industrial Wastes	8	5	3	5	7	5	5	2	8	20
Public Health	2	-	2	1	3	4	3	1	-	8
Refuse	--	1	-	4	7	1	4	6	1	12
Sewage	12	7	8	22	5	9	8	12	8	37
Stream Pollution	10	2	2	12	2	6	5	5	2	18
Water	10	2	9	12	1	7	7	11	5	30
Miscellaneous	3	3	4	9	-	-	4	5	3	12
Total	61	22	36	72	30	36	48	48	31	163
Per Cent of Total (rounded)	28	10	16	33	14	22	29	29	19	

SUMMARY OF SANITARY ENGINEERING PAPERS IN ASCE TRANSACTIONS 1949-1958

Year	Total Number of Transaction Papers	Average Number of Papers Per Division	Number of Sanitary Engr. Papers	Per Cent SA Papers of Total
1958	56	4	9	16
1957	62	4	7	11
1956	62	4	4	6
1955	61	5	5	8
1954	66	5	3	5
1953[a]	41	3	4	10
1953[b]	81	6	11	14
1952	58	4	5	9
1951	53	4	5	9
1950	35	3	2	6
1949	28	2	1	4
Total	603		56	9

[a] The regular 1953 issue, Volume 118.
[b] The special Centennial issue, Volume Ct.

SANITARY ENGINEERING RESEARCH

PHS RESEARCH GRANT AWARDS

The following list includes all awards made in the field of water pollution by the Public Health Service from July to December 1959. The list includes the name of the contracting organization, the Principal Investigator's name, and the title of the project. Further information concerning such grants may be obtained from Harry A. Faber, Engineering Resources Program, Public Health Service, Washington 25, D.C.

California

Humboldt State College, Arcata
John W. DeWitt, Assoc. Prof., Div. of Natural Resources
Improvement of oxidation pond effluents by fish-food organisms and fish.

National Canners Association, Berkeley
Walter A. Mercer, Bacteriologist
Water conservation in food canning.
Stanford University, Stanford
Joseph B. Franzini, Ph.D., Assoc. Prof. of Civil Engineering
Evaporation supression and effects on biota.
University of California, La Jolla
Galen E. Jones, Ph.D., Asst. Research Microbiologist
Effect of organic substances from sea on bacteria.
University of Southern California, Los Angeles
Robert C. Merz, Prof. of Civil Engineering
The chromatographic assay of extracellular products of algal metabolism.

District of Columbia

Georgetown University
Walter C. Hess, Ph.D., Asst. Dean for Research and Professor of Biological Chemistry
Application of a rapid isotope method for bacteriological determinations.

Florida

University of Florida, Gainesville
Taxonomy and Ecology of iron and sulfur bacteria.

Georgia

Georgia Institute of Technology, Atlanta
Dr. Werner N. Grune, Assoc. Prof. of Civil and Sanitary Engineering
Application of gas chromatography to sludge digestion gas analysis.

Illinois

Northwestern University, Evanston
John A. Logan, Sc.D., Chairman & Prof. Civil Engineering
Systems analysis of municipal sewage treatment.
University of Illinois, Urbana
T. E. Larson, Ph.D., Head, Chemistry Section
Corrosion and tuberculation in distribution systems.
Ben B. Ewing, Ph.D., Assoc. Prof. of Sanitary Engineering
Rate of synthetic detergents in soils and ground waters.
Ralph S. Wolfe, Ph.D., Assoc. Prof., Department of Bacteriology
The higher bacteria of polluted streams.

Indiana

Indiana University, Bloomington
Frank N. Young, Ph.D., Assoc. Prof. of Zoology
Taxonomy of beetles useful as indicators of pollution.
Purdue University, Lafayette
C. J. Goodnight, Ph.D., Professor
Oligochaetes as indicators of pollution.

Harvey R. Wilke, Assoc. Prof. of Sanitary Engineering
Control and treatment of farm pond water for domestic use on the farm.
G. H. Teletzke, Ph.D., Asst. Prof. of Sanitary Engineering
Factors affecting acid formation during sewage sludge digestion.

Iowa

Iowa State College, Ames
John L. Cleasby, Asst. Prof. of Civil Engineering
Selection of optimum filtration rates for sand filters.
State University of Iowa, Iowa City
Robert L. Morris, Asst. Prof. Hygiene & Preventive Medicine
Absorption and release of fluoride ion by zeolites.

Maryland

Johns Hopkins University, Baltimore
John C. Geyer, Chairman, Department of Sanitary Engineering
Low-flow augmentation for stream pollution abatement.

Massachusetts

Massachusetts Institute of Technology, Cambridge
Arthur T. Ippen, Ph.D., Prof. of Hydraulics
Mechanics of waste water diffusion.
J. Martin Symons, Sc.D.
The biochemistry of the synthesis in activated sludge.
Metcalf and Eddy, Boston
Clair N. Sawyer, Associate
Biochemical endproducts and methane formation.

Michigan

Michigan State University, East Lansing
Robert F. McCauley, Ph.D., Assoc. Prof.
Protective coatings for water distribution systems.
R. U. Byerrum, Ph.D., Assoc. Prof. of Chemistry
Toxicity studies on cadmium and hexavalent chromium in concentrations that may be expected in public water supplies.
Robert C. Ball, Ph.D.
A study of fundamental productivity in a stream.
University of Michigan, Ann Arbor
George H. Lauff, Ph.D., Instructor of Zoology
The role of substrate in stream fauna distribution.
Arthur D. Hasler, Prof., Department of Zoology
Support of International Congress of Limnology, 1962
J. A. Borchardt, Ph.D., Assoc. Prof. of Civil Engineering
The use of algae in the treatment of sewage effluents.

Missouri

Washington University, St. Louis
D. W. Ryckman, Sc.D., Assoc. Prof. of Sanitary Engineering
Characterization of extracts from the Missouri River.

Nebraska

University of Nebraska, Lincoln
Richard Y. Morita, Ph.D., Department of Bacteriology
Study of beggiatoa species.

New Hampshire

University of New Hampshire, Durham
L. W. Slanetz, Ph.D., Prof. of Bacteriology
Enterococci as indicators of water pollution.

New Jersey

Rutgers University, New Brunswick
H. Heukelekian, Ph.D., Chairman, Department of Sanitation
Effect of loading and other variables on the physical and biochemical properties of activated sludge.
David Pramer, Ph.D., Assoc. Prof., Dept. of Ag. Microbiology
An evaluation of factors contributing to the bactericidal effect of sea water.

New Mexico

New Mexico College of A & MA, Las Cruces
John W. Clark, C.E., Assoc. Prof. of Civil Engineering
Aerobic digestion in the liquid state.

New York

New York University, New York
William E. Dobbins, Sc.D., Prof. of Sanitary Engineering
Gas transfer to and from turbulent liquids.

North Carolina

University of North Carolina, Chapel Hill
Charles M. Weiss, Ph.D., Assoc. Prof. of Sanitary Science
Use of fish to detect organic insecticides in water.
Daniel A. Okun, Sc.D., Prof. of Sanitary Engineering
Oxygen in waste treatment, effect of surfactants.

Ohio

Kenyon College, Gambier
James M. Pappenhagen, Ph.D., Asst. Prof. of Chemistry
Determination of ions in wastes and waters.
Ohio State University, Columbus
Wm. MacNevin, Ph.D., Prof. of Chemistry
Fluoride-triggered electrode reactions.
Chester I. Randles, Ph.D., Assoc. Prof. of Bacteriology
Physiology of aquatic bacteria pertinent to acid mine waste problem.
H. H. Weiser, Ph.D., Prof., Department of Bacteriology
Microbiological aspects of farm-pond water supplies.

University of Cincinnati, Cincinnati
John D. Eye, Assoc. Prof., Department of Civil Engineering
Continuous measurement and recording of B.O.D. data.
Robert Lemlich, Ph.D., Asst. Prof., Chemical Engineering
Foam fractionation with reflux.

Oklahoma

Oklahoma State University, Stillwater
Troy C. Dorris, Ph.D., Prof. and Dir. of Wildlife Management
Continuous-flow bioassay of oil refinery effluents.

Oregon

Oregon State College, Corvallis
Fred Merryfield, Prof. of Sanitary Engineering
Study of the physical, chemical and biological aspects of deep trickling filters.
Chas. E. Warren, Asst. Prof., Dept. of Fish and Game Management
Ecological studies of a polluted experimental stream.
Harry Freund, Ph.D., Assoc. Prof. of Chemistry
Chemistry and toxicology of complex cyanides in aqueous solutions.
Roland E. Dimick, Prof. of Fish and Game Management
An experimental study of the resistance of representative marine animals to various pollutional conditions in marine environments.

Pittsburgh

University of Pittsburgh, Pittsburgh
M. A. Shapiro, Assoc. Prof. of Sanitary Engineering
Chemical and physical studies of sewage sludge dewatering.

Pennsylvania

University of Pittsburgh, Pittsburgh
Richard C. Dugdale, Ph.D., Instructor in Zoology
Primary nitrogen fixation in natural waters.
A. G. Kammer, M.D., Head, Department of Occupational Health
Biochemical classification of toxic agents.

Rhode Island

University of Rhode Island, Kingston
Chas. J. Fish, Ph.D., Prof. of Marine Biology
Environmental relationships of benthos in salt ponds.

Texas

Baylor University, Waco
Floyd F. Davidson, Ph.D., Prof. of Biology
Toxic properties of blue-green algae.
Texas Technical College, Lubbock
Vernon W. Proctor, Ph.D., Asst. Prof. of Biology
Quantitative estimate of fatty acids in algal cultures.

Utah

University of Utah, Salt Lake City
Arden R. Gaufin, Ph.D., Assoc. Prof. of Zoology
Bio-assays to determine the toxicity of pesticides to aquatic invertebrates.
Utah State University, Logan
Wm. F. Sigler, Ph.D., Head, Department of Wildlife Management
The physiology of fluorosis in fresh-water fish.

Vermont

University of Vermont, Burlington
Milton Potash, Ph.D., Asst. Prof. of Zoology
Nutritional requirements of Kirchneriella subsolitaria.

Washington

The State College of Washington, Pullman
Rolf T. Skrinde, Assistant Sanitary Engineer
Effect of irrigation use on quality of return waters.
University of Washington, Seattle
Richard H. Bogan, Sc.D., Assoc. Prof. of Civil Engineering
A study of the biochemical behavior of pesticides in natural waters.
Robert O. Sylvester, Prof. of Sanitary Engineering
Character and significance of irrigation return flows.

Wisconsin

University of Wisconsin, Madison
Geo. P. Fitzgerald, Ph.D., Research Assoc.
The effect of algae on B. O. D. measurements.
Gerad A. Rohlich, Ph.D., Director, Hydraulic and Sanitation Laboratory
Hydraulic characteristics of circular sedimentation basins.
John C. Neess, Ph.D., Assoc. Prof. of Zoology
Primary nitrogen fixation in natural waters.

* * *

WATER SUPPLY & POLLUTION CONTROL

TWO LARGE SANITARY ENGINEERING PROJECTS NOMINATED FOR "OUTSTANDING CIVIL ENGINEERING ACHIEVEMENT OF 1959" AWARD

Allegheny County's new sewage disposal system, at Pittsburgh, and the Torresdale Water Treatment Plant, Philadelphia, have been nominated for the "Outstanding Civil Engineering Achievement of 1959," the American Society of Civil Engineers announced recently.

These are two of 12 nominations made throughout the country for the outstanding civil engineering achievement. Judging will be by a jury of engineering magazine editors, and the winner will be announced by the Society's Board of Direction at ASCE's national convention in New Orleans in March.

The Allegheny system was built at a cost of $100 million, and serves Pittsburgh and 69 surrounding communities. The project involves 69 miles of intercepting sewers, six sewage lift stations and a central sewage

treatment plant having a rated capacity of 150 million gallons daily, with a maximum capacity of 300 million gallons per day. It serves not only the needs of the area's tremendous industrial activity, but also the 1,400,000 residents of Pittsburgh and its environs.

In a citation accompanying this nomination, it was stated that the most significant engineering contribution resulting from the project was the extensive investigation, subsequent proof and application of an unique method of sludge concentration and incineration.

Torresdale was built at a cost of $25,000,000 to serve the city of Philadelphia's 2,200,000 inhabitants. One of America's two largest water treatment plants, it is located on a site overlooking the tree-lined Delaware River. Known as a "push button" water treatment plant, it is interlaced with numerous automatic and semi-automatic controls for water flow, chemical feeding filtration rate and the use of its facilities.

APATHY HINDERS POLLUTION ABATEMENT, ENR SAYS

Engineering News-Record, in a recent editorial, called public apathy the first big hurdle in overcoming pollution abatement.

"Ruin of this country's streams by pollution becomes a more ominous threat each day," the News-Record commented.

The publication then quoted Assistant Surgeon-General Mark Hollis of the Public Health Service as saying "we ought to spend something over a half billion dollars a year to build treatment plants, instead of the $400 million we are spending now." The reason is that treatment plant construction is not keeping pace with the new demand created by additional sewers and population growth. The publication then called for action to sell the public on pollution abatement needs.

"It's a tough selling job," the editorial said. "For one thing, you are trying to get people to pay for something that they may have been getting along perfectly well without. Secondly, you are trying to persuade a segment of the population to buy something that in all likelihood will redound more to the direct benefit of others (people downstream) than to themselves. Thirdly, a great part of the return on the investment may appear to be on the somewhat negative benefit of forestalling an evil rather than the immediate achievement of a positive good."

The editorial concluded with the thought: "The first big hurdle is to overcome public apathy. It is the biggest deterrent to a successful clean-streams program."

WASHINGTON METROPOLITAN SILT CONFERENCE

On November 24 the Potomac River Commission sponsored a conference on metropolitan silt pollution control, the first such conference to be held in the Washington area. Every speaker at the conference seemed to agree with Dr. Abel Wolman's thesis that "Silt is the worst pollutant of the Potomac River." Except for that agreement, there seemed to be unanimity of opinion on only one other point; the lack of fact upon which to base effective programs for the control of siltation. Major concern in the Washington metropolitan area seems to stem from the transition of once rural areas to urban ones. One significant indication of this was that silt surveys of Lake Barcroft in Fairfax County, Virginia show that the lake is currently silting-up at a rate

eight times faster than in 1938. This alarming increase has been caused by the rapid urbanization of two small watersheds both of which empty into the lake. In order to save the lake from eventual destruction, costly measures are being initiated. Two catch basins for sediment will soon be built where these two small streams enter the lake at a cost of $180,000. Another example of this trend in increased siltation from urbanization is on the Anacostia River where a 12 million dollar flood control and navigation project was made necessary by siltation of the Anacostia watershed. Maintenance of the project is expected to call for the removal by dredging of 80,000 cubic yards of silt a year at an annual cost of $60,000.

A condensation of the proceedings at the Metropolitan Silt Control Conference will be published soon by the Interstate Commission on the Potomac River Basin. Further information concerning these proceedings may be obtained by writing the Commission at 203 Transportation Building, Washington 4, D.C.

OCEAN DISPOSAL OF SEWAGE

Paul R. Bonderson, Executive Officer, California State Water Pollution Control Board reports that two-thirds of California's sewage and industrial waste effluent is discharged to salt waters. At the same time, rapidly growing population is also demanding greater recreational use of these estuarine and coastal waters. In the field of marine disposal of wastes there seems to be a need for a simple biological method of evaluating the effect of waste discharges before conditions become so adverse that they are apparent to anyone and extremely difficult to correct. A research program in this field is being conducted for the State Board by the Allan Hancock Foundation of the University of California. Results of the first two years of study have been published and distributed recently as State Board Publication No. 20, "Oceanographic Survey of the Continental Shelf Area of Southern California." Information concerning this publication may be obtained from Mr. Bonderson, State Water Pollution Control Board, Room 316, 1227 O Street, Sacramento 14, California.

NEW WASTE CONTROL MEASURES TO BE TAKEN BY KANAWHA VALLEY INDUSTRIES

The West Virginia Water Resources Commission reports that major industries operating on the Kanawha River have agreed to take concerted action to reduce oxygen-consuming wastes loads by 40 per cent by Spring 1963. The reduction will be based upon 1958-1959 waste loads.

A river survey, conducted during the Summer of 1958 with the cooperation of the Valley industries, paved the way for the waste reduction program now in progress. Another important step was the formation, at the Commission's request, of the Kanawha River Industrial Advisory Committee. The purpose of this committee is to advise the Commission on technical matters pertaining to the abatement of complex industrial wastes on the Kanawha River.

Achievements to date include a standardized and more comprehensive periodic waste load report, a revised permit system and a phased waste reduction program, the first phase of which is to be completed by the Spring of 1963.

A second survey is scheduled for the Summer of 1964 which will determine

the effectiveness of the first phase waste reduction program and determine the extent of further cut-backs. The Commission plans to then establish a minimum dissolved oxygen level for each stretch of the Kanawha River. The final goal is an oxygen content of at least 3 parts per million. A 30-35 mile stretch below Charleston is now completely devoid of oxygen during the low stream flow periods of late summer and fall.

NEW HAMPSHIRE AIDS SEWAGE TREATMENT CONSTRUCTION

The Governor of New Hampshire recently approved House Bill No. 6 to provide aid to municipalities for water pollution control. The new measure provides for an annual State payment to qualified municipalities 20 percent of the yearly amortization charges on the original costs of acquisition and construction of sewage disposal facilities. The bill is retroactive, and includes municipalities which have undertaken such construction since July 1, 1947. The Act appropriates $68,000 for F.Y. 1960, and $96,000 for F.Y. 1961, plus additional amounts for administration.

LOW INTEREST COST WATER SYSTEM LOANS SUGGESTED

A member of the Maine Public Utilities Commission recommended to the Senate Select Committee on Water Resources at its hearing in Augusta that it considers the possibility of making low-interest rate loans for constructing municipal water systems. Richard J. McMahon suggested that the difference between a 2 percent federal loan and a 4-1/2 percent private loan might be the margin that would prevent many small villages from building water systems.

Maine has about 70 villages with over 750 population that do not have water systems. McMahon expressed doubts that many would have courage to incur such expenditures unless the over-all cost of providing service could be reduced. He cited the loans presently available for electric cooperatives from the Rural Electrification Administration as the type loan he was suggesting.

RECENT CROSS - CONNECTION INCIDENTS

In Crestview, Florida recently a plumber erroneously connected a 250 gallon butane tank to a water pipe in a cafe. One explosion wrecked the cafe and two others damaged private homes. Five persons were injured, three requiring hospitalization. The town's population spent a night of terror while Water and Fire Department personnel flushed water mains and service pipes.

In Atlanta, Georgia blood instead of water started forth from the drinking fountain in an undertaking establishment. A water operated aspirator used in embalming became plugged and the water pressure went low. A small water line feeding too many outlets caused the pressure drop. City Water and Health Departments required the elimination of the cross connection in addition to installing a larger meter and requiring larger service lines. In this particular case there was no evidence that the water supply outside of the building was affected.

The water supply of a school in Center Point, Alabama recently became contaminated from a cross connection occurring during the cleaning of the school septic tank. Water contaminated with septic sewage was first noted in the kitchen of the school as personnel were preparing lunch for the school

children. The water drawn was brown in color and of a very strong odor. The motor used to operate the pump for cleaning the septic tank was water cooled. A connection from the school's water supply was made to the motor. The same line was used to prime the pump when beginning operation. In this case the operator failed to close the valve between the pump and the water line and a check valve installed in the line failed to operate satisfactorily. As a consequence, while sludge was being pumped from the septic tank into a tank truck, it was also being pumped back through the water supply line into the school system. Health authorities from the Jefferson County (Birmingham) Alabama Health Department took immediate preventive action to forestall any epidemics developing among the school children. Their efforts were totally satisfactory.

EVAPORATION TESTS CONTINUE IN WEST

U.S. Bureau of Reclamation personnel are continuing water evaporation research in experiments on Lake Mead and Lake Sahuaro. The new tests are to determine the behavior of the chemicals hexadecanol and octodecanol under various wind and other climatalogical conditions. The odorless and colorless chemicals form a thin film on water surfaces which retards evaporation. An estimated 13 million acre-feet of water is lost annually from western lakes due to evaporation.

WATER SAMPLING SYMPOSIUM AT SEC

The Public Health Service announces a symposium on water quality data collection and utilization, to be held at the Sanitary Engineering Center at Cincinnati, Ohio, on August 23, 24, and 25, 1960.

The symposium will bring together representatives of organizations currently engaged or interested in establishment, instrumentation, and operation of water sampling networks and other related activities in water resources management. It will provide an opportunity to exchange information on objectives, organization, and operation of such networks; on the analysis and utilization of data; and on existing needs in the field of water quality measurement. Particular emphasis will be placed on automatic instrumentation.

When ready for distribution the program will be sent to organizations and individuals known to be interested in this field. Anyone wishing to receive the agenda and other announcements may write to The Director, Robert A. Taft Sanitary Engineering Center, 4676 Columbia Parkway, Cincinnati 26, Ohio.

TWO NEW OFFICE OF SALINE WATER RESEARCH AND DEVELOPMENT PROGRESS REPORTS NOW AVAILABLE

Research and Development Progress Report No. 29, "Producing Permanently Hydrophylic Surfaces on Plastic Films for Solar Stills," and Research and Development Progress Report No. 30, "Weathering Tests of Plastics and Design of Suspended Envelope Solar Stills," have been published and are for sale by the U.S. Department of Commerce, the Department of the Interior announced today.

Report No. 29 is a report of experimental work conducted by the Franklin Institute of Philadelphia, Pa., under contract with the Department of the

Interior's Office of Saline Water, to develop the means of producing water-wettable surfaces on plastic films of an essentially permanent nature for use as the transparent cover for condensing surfaces on solar stills.

Solar stills constructed with plastic materials may substantially reduce capital investment, thereby reducing the cost of the product water. The development of a hydrophylic surface on the plastic will improve fresh water production rates by promoting sheet formation rather than drop formation of the condensed water. Increased efficiency is obtained by increased transmission of solar radiation and by the elimination of dropback of distilled water into the evaporation basin.

The hydrophylic surfaces developed by this contract will provide samples for pilot scale testing as solar still condensing surfaces at the Office of Saline Water's Solar Research Station at Daytona Beach, Florida.

The report of a program for investigation of the resistance of plastic film materials to solar radiation and weathering under distillation conditions is contained in Research and Development Progress Report No. 30. This investigation, which also included preliminary design, construction, and initial testing of suspended envelope stills, was carried out by the Bjorksten Research Laboratories, Inc., of Madison, Wisconsin, under contract with the Office of Saline Water.

Recent developments in plastics technology has permitted the use of thin films of transparent plastics in the construction of solar stills. Information obtained on the resistance of various plastic film materials to solar radiation under distillation conditions, test results, construction experience, and designs of prototype plastic envelope stills, are contained in this report.

Copies of these reports are for sale by the U.S. Department of Commerce, Office of Technical Services, Washington 25, D.C. The price of Progress Report No. 29 is $0.75. To order copies of this report specify order number PB 161064. The price of Progress Report No. 30 is $1.50, and the order number is PB 161095. Make check or money order payable to the Office of Technical Services, Department of Commerce.

SOUTHERN MUNICIPAL AND INDUSTRIAL WASTE CONFERENCE

The Ninth Southern Municipal and Industrial Waste Conference is scheduled for April 7-8, 1960 at North Carolina State College in Raleigh, North Carolina. The program will be keynoted by Dr. Clair N. Sawyer, Director of Research for Metcalf and Eddy, with the topic "Stop, Look, and Listen."

Dr. Mark D. Hollis, Chief Sanitary Engineer, U.S. Public Health Service will deliver the April 7th banquet address. His topic is "The Future is Now."

The program will feature papers directed at encouraging Industries and Municipalities to work independently and collectively toward the solution of their waste problems.

Among the speakers who will describe problems, techniques, and successful waste treatment methods will be J. W. Masselli of Wesleyan University, Connecticut, W. D. Barlow of L. E. Wooten & Company, North Carolina, Mr. Fred Doutt of Champion Paper and Fiber Company, Professor H. D. Cook of N. C. State College, Mr. David Milne of General Motors Corp., Detroit, Mr. Clayton Billings of Public Works Magazine and Mr. Arthur Smith of Chatham Manufacturing Company.

Information is available from Professor Charles Smallwood, Jr., Mann Hall, North Carolina State College.

CALIFORNIA REPORT ON QUANTITY OF OILY SUBSTANCES ON BEACHES

The California State Water Pollution Control Board recently announced release of its Publication Number 21 entitled (Part One) "Determination of the Quantity of Oily Substances on Beaches and in Near-Shore Areas and (Part Two) Characterization of Coastal Oil Pollution by Submarine Seeps." Part One of the report was prepared under contract for the State Board by the Sanitary Engineering Research Laboratory at the University of Southern California. Part Two was prepared by the Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio.

Copies of this publication may be purchased from the California State Printing Division, Documents Section, Sacremento 14, California, at a cost of $1.50 per copy. Remittance must accompany order.

SUMMARY OF SALINE WATER CONVERSION PROGRAM

The Senate Select Committee on National Water Resources recently released Committee Print #26 entitled "Saline Water Conversion." Senator Robert S. Kerr, Chairman of the Select Committee requested the Office of Saline Water, U.S. Department of the Interior to prepare the report on an evaluation of recent efforts in the field of desalinization of brackish and ocean waters. Interior's report to the Committee formed the basis for this publication.

This report presents a rather detailed explanation of the fundamental research conducted by the Office of Saline Water since its beginning seven years ago. In addition, the various processes developed are covered in some detail as well as the demonstration plant program now going forward in several parts of the country. While not highly technical, the report does present a very useful summary as to the work accomplished and the present status of the field of desalinization.

At the time that Senator Kerr requested this report from the Department of the Interior, he also requested the Atomic Energy Commission to prepare a report on the application of nuclear energy for desalinization purposes. This will be published later as Committee Print #27.

Information concerning the availability of both these reports may be obtained from Mr. Theodore M. Schad, Staff Director, Senate Select Committee on National Water Resources, Senate Office Building, Washington 25, D.C.

NEW WATER POLLUTION CONTROL ADVISORY BOARD APPOINTMENT

President Eisenhower recently announced the appointment of John Charles Daly, widely known T.V. and radio commentator and Vice President of the American Broadcasting Company, for a three-year term as a member of the Water Pollution Control Advisory Board. This Board which is advisory to the Surgeon General of the Public Health Service in conducting the Federal Water Pollution Control Program under P.L. 660 usually meets semi-annually. Mr. Daly's appointment brings the Board to full strength of nine members. A complete list was given in the January 1960 issue of this Newsletter.

NATIONAL WILDLIFE WEEK THEME "WATER—KEY TO YOUR SURVIVAL"

National Wildlife Week, sponsored annually by the National Wildlife Federation and its affiliates throughout the country, this year features the theme of water problems. The observance slogan is: "Water—Key to Your Survival." The 1960 observance is scheduled for the period March 20-26, and the National Honorary Chairman this year is again Walt Disney.

Kits and other promotional material concerning this observance are available from the National Wildlife Federation, 232 Carroll Street, Northwest, Washington 12, D.C.

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NUCLEAR ENERGY

STRONTIUM 90 IN MAJOR U.S. WATERS

The Public Health Service recently reported on the levels of strontium 90 in the Nation's major water courses.

The levels continue to be far below the level of 80 micromicrocuries per litre recommended by the National Committee on Radiation Protection as permissible for life time exposure of the general population to strontium 90.

The figures, based on weekly samples from 51 stations on 17 major rivers, are part of the information being gathered on stream pollution by the national water quality network operated by the Service.

The attached table shows the strontium level in terms of micromicrocuries per litre for the four quarters of 1959 for most of the stations of the network. Data from a few are either being rechecked or are not presently available.

	STRONTIUM ACTIVITY, uuc/liter			
SAMPLING POINT	1st Qtr	2nd Qtr	3rd Qtr	4th Qtr
Great Lakes				
Duluth, Minn.	0.7	0.7	0.1	0.2
Gary, Ind.	0.7	0.7	1.0	0.4
Detroit, Mich.	0.5	0.5	0.4	0.6
Buffalo, N.Y.	1.1	1.1	1.3	0.8
No. Atlantic Coastal Rivers				
Hagerstown, Md. (Potomac)	-	1.4	1.3	0.7
Great Falls, Md. (Potomac)	2.4	2.4	1.0	0.8
Easton, Pa. (Delaware)	-	-	2.0	-
Philadelphia, Pa. (Delaware)	0.8	0.8	1.1	0.6
Poughkeepsie, N.Y. (Hudson)	4.5	4.5	1.1	1.0
Lowell, Mass. (Merrimack)	1.6	1.6	2.1	1.2

STRONTIUM ACTIVITY, uuc/liter

SAMPLING POINT	1st Qtr	2nd Qtr	3rd Qtr	4th Qtr
No. Atlantic Coastal Rivers (cont.)				
Northfield (Amhurst) Conn. (Connecticut)	-	-	-	1.9
Savannah River				
No. Augusta, S.C.	0.6	0.6	2.8	0.3
Savannah, Ga.	1.4	1.4	1.9	0.7
Tennessee River				
Chattanooga, Tenn.	-	4.7	4.8	3.8
Rio Grande River				
El Paso, Texas	0.7	0.7	1.1	0.3
Laredo, Texas	0.3	0.3	1.1	0.2
Colorado River				
Loma, Colo.	2.9	2.9	0.7	0.3
Hoover Dam, Ariz.-Nev.	1.8	1.8	1.2	-
Parker Dam, Ariz.-Calif.	0.9	0.9	2.7	0.6
Yuma, Ariz.	0.7	0.7	1.6	0.8
Alsea River				
Alsea, Ore.	0.0	0.0	0.2	0.2
Arkansas River				
Coolidge, Kans.	2.4	2.4	1.7	1.4
Poca City, Okla.	1.6	1.6	2.4	0.5
Pendleton Ferry, Ark.	-	-	2.2	0.3
Fort Smith, Ark.	-	-	-	2.1
Snake River				
Weiser, Id.	0.4	0.4	0.4	-
Wawawai, Washington	0.5	0.5	0.9	0.4
Columbia River				
Wenatchee, Washington	0.6	0.6	0.8	0.5
Pasco, Washington	1.4	1.4	1.2	1.0
Bonneville Dam, Wash.-Ore.	0.9	0.9	1.1	0.5
Beaver Army Term., Ore.	0.7	0.7	1.2	0.5
Red River				
Denison, Texas	2.0	2.0	2.3	1.0
Index, Ark.	-	-	2.5	0.6
Alexandria, La.	0.3	0.3	2.2	0.4
Ohio River				
E. Liverpool, Ohio	0.9	0.9	2.1	1.2
Huntington, W. Va.	0.8	0.8	1.1	1.0
Cincinnati, Ohio	0.6	0.6	0.9	0.9
Evansville, Ind.	1.8	1.8	1.6	2.0
Cairo, Ill.	1.1	1.1	2.9	1.2

STRONTIUM ACTIVITY, uuc/liter

SAMPLING POINT	1st Qtr	2nd Qtr	3rd Qtr	4th Qtr
Missouri River (Incl. Yellowstone R.)				
Sidney, Mont. (Tr)	0.8	0.8	1.9	0.7
Williston, N.D.	1.6	1.6	1.4	0.5
Bismarck, N.D.	0.4	0.4	1.5	0.8
Yankston, S.D.	0.8	0.8	2.2	0.4
Omaha, Neb.	0.5	0.5	2.1	1.1
St. Joseph, Mo.	0.7	0.7	1.3	0.6
Kansas City, Kans.	0.8	0.8	1.7	0.3
St. Louis, Mo.	0.7	0.7	0.6	0.5
Mississippi River				
Red Wing, Minn.	-	-	1.6	0.5
Dubuque, Ia.	1.1	1.1	1.5	1.6
Burlington, Ia.	1.1	1.1	2.1	1.1
E. St. Louis, Ill.	1.4	1.4	3.4	1.2
Cape Girardeau, Mo.	1.2	1.2	1.6	1.2
W. Memphis, Ark.	0.9	0.9	2.1	1.3
Vicksburg, Miss.	1.0	1.0	-	-
New Orleans, La.	1.2	1.2	0.8	1.0
Delta, La.	-	-	-	1.3

NEW GAMMA RADIATION DETECTION DEVICE

More efficient method for detecting hazardous gamma radiation is a system of increasing radiation sensitivity of glass, developed by Armour Research Foundation. Method uses long glass fibers, specially coated with lower refracting glass to give constant surface absorption. Exposure to radiation darkens fibers, permitting immediate reading. Fibers can be coiled into small spool or disk and worn by laboratory and military personnel to measure exposure to gamma radiation. Further information can be obtained from Armour Research Foundation of Illinois Institute of Technology, Chicago 16, Illinois.

PHS SETS UP TWO RADIOLOGICAL LABORATORIES

The Public Health Service Division of Radiological Health, has recently set up two branch radiological facilities. The first of these is located in the western part of the United States at Las Vegas, Nevada and the second was dedicated on February 11 in Montgomery, Alabama. These two facilities are being developed to furnish laboratory services as well as to serve as training facilities for State Health Departments and other regulatory authorities in these areas.

The Southwestern facility at Las Vegas, Nevada is presently housed in temporary quarters while negotiations are under way to secure permanent headquarters on the University of Nevada campus at Las Vegas. Formal proposals are now being submitted to the University's Board of Regents for subsequent transmittal to the Nevada State Legislature. Present plans call

for a permanent building to be occupied in the fall of 1960. The Southwestern facility is located in laboratories vacated by the Public Health Service's Communicable Disease Center near Gunter Air Force Base in Montgomery.

FIRST INTERNATIONAL CONFERENCE ON RADIOACTIVE WASTE DISPOSAL

The first international conference on radioactive waste disposal was held recently in Monaco. This conference, a joint venture of International Atomic Energy Agency and NESCO, was attended by over 300 experts from 32 countries. The meeting showed that many of the differences of opinion about approaches to waste disposal were founded less in real difference of principle or philosophy than in differences of definition. Some important clarifications on these were reached at the conference.

PHS INAUGURATES NEW RADIATION PUBLICATION

Secretary Arthur S. Fleming of the Department of Health, Education, and Welfare announced at his press conference on January 12 that the Public Health Service will initiate, on March 1, a monthly publication of data on radiation. This will contain information derived from the agencies of the DHEW and, as developed, from other departments and agencies of the Federal Government, State Health Departments, and other sources. Whenever possible, the data will be analyzed and interpreted; however the periodical will also contain raw data, so labeled, which will be of use to the scientific community. Each third issue, beginning with the July 1 issue, will contain analyses based on information collected during the preceding quarter or longer period.

Further information concerning the availability of this publication may be obtained by writing to Chief, Division of Radiological Health, Public Health Service, Washington 25, D.C.

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AIR POLLUTION

AIR QUALITY AND MOTOR EXHAUST STANDARDS ADOPTED BY CALIFORNIA

At their meeting in Berkeley on December 4, the State Board of Public Health adopted the standards for air quality and motor vehicle exhaust drafted by the State Department of Public Health to combat air pollution in California.

The 1959 Legislature issued a mandate to draft statewide standards for air cleanliness and motor vehicle exhaust emission for submission to the State Board of Public Health and, after board adoption, for submission to the Governor and the Legislature before February 1, 1960.

The air quality standards of motor vehicle exhaust standards now set represent the first effort in the nation to attack the air pollution problem by setting allowable concentrations of pollutants for control action and the first effort to set standards for pollutants emitted from motor vehicle exhaust.

The air quality standards reflect the relationship between the intensity and composition of air pollution and the health and comfort of human beings, as well as the damage to vegetation and the interference with visibility.

In order to reflect this relationship the State Department of Public Health has designated three levels of air pollution conditions. The levels are:

Pollutant	"Adverse" level	"Serious" level	"Emergency" level
	Level at which there will be sensory irritation, damage to vegetation, reduction in visibility or similar effects.	Level at which will be alteration of bodily function or which is likely to lead to chronic disease.	Level at which it is likely that acute sickness or death in sensitive groups or persons will occur.
Oxidant	Oxident index 0.15 part per million for 1 hour by the potassium iodide method.	---	---
Sulfur dioxide	1 ppm for 1 hour, or 0.3 ppm for 8 hours.	5 ppm for 1 hour.	10 ppm for 1 hour.
Carbon monoxide	---	30 ppm for 8 hours or 120 ppm for 1 hour.	---
Particulates	Sufficient to reduce visibility to less than 3 miles in the absence of fog.	---	---

In addition to the elements cited above, sulfuric acid, ozone, ethylene, nitrogen dioxide, lead, and other substances will be added to the standards after their effects upon health, vegetation, and visibility have been determined.

The motor vehicle exhaust standards are based on the air quality standards. They were arrived at by taking into consideration the existing air quality in Los Angeles County and the estimated emission from motor vehicles in that county by 1970. The exhaust standards would limit the emission of hydrocarbons to 275 parts per million by volume and the emission of carbon monoxide to 1.5 percent by volume. For motor vehicles to come within these limits would require a reduction of 80 percent in the emission of hydrocarbons and a 60 percent reduction in the emission of carbon monoxide.

PERSONNEL CAN NOW TEST 18 AIR POLLUTANTS IN 90 MINUTES WITH LOW-COST PORTABLE FIELD KIT

Kem-Tech Laboratories of Baton Rouge has developed a small, lightweight field test kit for both sampling and analysis of the atmosphere. Portable, easy to operate, and powered from an automobile cigarette lighter socket, it should enable personnel with a minimum of chemical training to conduct air pollution surveys at a modest cost.

The new kit can sample the air and analyze for some 10 particulates and 8 gases in less than an hour and a half. Although no provision is made for

analyzing them in the field, it can also sample organic compounds. It will be manufactured by Central Scientific Company and should be available by year's end for $400 or less. A prototype is already at hand at SEC, Cincinnati.

PENNSYLVANIA GETS AIR POLLUTION CONTROL LAW

Governor Lawrence of Pennsylvania recently signed into law a bill providing for state-wide air pollution control through a one-year study of conditions leading to the adoption of control standards.

The law will be administered through the Pennsylvania State Department of Health and an eleven-member commission to be appointed by the Governor. This commission will be established within the State Health Department with regulatory powers over air pollution conditions throughout the State. The administration of the law will be by the Division of Occupational Health.

The new law also provides for the appointment of Regional Air Pollution Control Associations by the Governor to rectify air pollution problems in local areas. This action is to be on a voluntary basis by "persuasion and conciliation." If these groups cannot stop pollution on this basis, the State Commission will have authority to enter and enforce control through the State Health Department.

During the next year's study, the Commission will hold public hearings as part of its survey. Following completion of the over-all survey and the public hearings, the Commission will develop rules and regulations for the future.

Under terms of the law, atmospheric pollution is subject to a summary fine of $30.00 to $300.00 or ten to 30 days in jail for first and second offenses. Chronic violators could be fined from 500 to 1,000 and jailed up to one year in the discretion of the court.

ASCE MANUAL STUDIES SANITARY LANDFILLS

The depositing of waste products on land is probably the oldest known method of waste disposal. The scientific development of this method in the United States, however, has taken place only in recent years. A review of the sanitary landfill method of refuse disposal was undertaken, in 1954, by a committee of the Sanitary Engineering Division of ASCE. The results of that committee's work are discussed in ASCE Manual of Engineering Practice No. 39, entitled "Sanitary Landfill."

The purpose of this new ASCE manual is to bring together the desirable and accepted practices involved in the operation of a satisfactory landfill. The manual includes a definition of this method of refuse disposal, which was used as the criterion in selecting and preparing the material to be covered. It should serve as a guide to those who desire to conduct an acceptable operation.

Copies of Manual No. 39 can be ordered from ASCE by use of the following coupon. The list price is $2.00 and Society members are entitled to a 50% discount.

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